สมบัติทางวิศวกรรมและกายภาพของดินที่ผุพังอยู่กับที่จากพื้นที่เสี่ยงภัยแผ่นดินถล่มในอำเภอ เชียงกลาง จังหวัดน่าน ประเทศไทย

นายธนกฤต ทองขาว

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิทยาศาสตรมหาบัณฑิต
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ENGINEERING AND PHYSICAL PROPERTIES OF RESIDUAL SOIL FROM LANDSLIDE HAZARD AREA IN AMPHOE CHIANG KLANG, CHANGWAT NAN, THAILAND

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A Thesis Submitted in Partial Fulfillment of the Requirements
for the Degree of Master of Science Program in Earth Science
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Thesis Title	ENGINEERING AND PHYSICAL PROPERTIES OF RESIDUAL SOIL FROM LANDSLIDE HAZARD AREA IN AMPHOE CHIANG KLANG, CHANGWAT NAN, THAILAND
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ธนกฤต ทองขาว : สมบัติทางวิศวกรรมและกายภาพของดินที่ผุพังอยู่กับที่จากพื้นที่เสี่ยงภัยแผ่นดินถล่มในอำเภอเชียงกลาง จังหวัดน่าน ประเทศไทย.(ENGINEERING AND PHYSICAL PROPERTIES OF RESIDUAL SOIL FROM LANDSLIDE HAZARD AREA IN AMPHOE CHIANG KLANG, CHANGWAT NAN, THAILAND) อ.ที่ปรึกษาวิทยานิพนธ์หลัก: รศ.ดร. มนตรี ชูวงษ์, อ.ที่ปรึกษาวิทยานิพนธ์ร่วม : รศ.ดร. ปัญญา จารุศิริ, 197 หน้า.

งานวิจัยที่เกี่ยวข้องกับดินถล่มในประเทศไทยได้มุ่งเน้นไปที่การจัดโซนพื้นที่อันตรายจากการเกิดดินถล่ม โดยทำการวิเคราะห์ข้อมูลอยู่บนปัจจัยพื้นฐานต่างๆ เช่น สภาพภูมิประเทศ, การใช้ประโยชน์ที่ดิน,มุมความชันของลาดดินธรรมชาติ, ปริมาณน้ำฝนที่มากเกินกว่าปกติ ฯลฯ ดังนั้นงานวิจัยจึงตั้งวัตถุประสงค์เพื่อบูรณาการข้อมูลทางธรณีวิทยา ,การทดสอบและผลการศึกษาสมบัติทางวิศวกรรมของดินที่ผุพังมาจากหินต้นกำเนิดซึ่งประกอบไปด้วย สมบัติทางกายภาพของดิน สมบัติทางด้านกำลังรับแรงเฉือนของดินที่ลดลงตามการเพิ่มขึ้นของความชื้นในดิน พื้นที่ศึกษาบ้านหนองปลา อำเภอเชียงกลาง จังหวัดน่าน พบหินตะกอนเนื้อเม็ดที่เป็นหินต้นกำเนิด ได้แก่ หินดินดาน หินทรายและหินทรายแป้ง ดินและหินผุเหล่านี้มีอัตราการผุพังสลายตัวสูงจึงทำให้มีความอ่อนไหวต่อการเกิดดินถล่ม ผลการศึกษาพบว่า ค่าเฉลี่ยความชื้นในดิน ค่าเฉลี่ยความถ่วงจำเพาะของดิน ค่าเฉลี่ยขีดจำกัดเหลวและค่าเฉลี่ยขีดจำกัดพลาสติก มีค่าเท่ากับ 24.83 %, 2.68 , 44.93 % และ 29.35 % ตามลำดับ ดัชนีพลาสติกมีค่าเท่ากับ 15.58 % ค่าความซึมน้ำของดินอยู่ระหว่าง 9.36E-06 ถึง 6.81E-07 เซนติเมตรต่อวินาที ตามลำดับ ดินในพื้นที่ศึกษาสามารถจำแนกโดยใช้ระบบการจำแนกดินทางวิศวกรรมแบบเอกภาพ (USCS) ได้เป็น ML-CL, CL-ML, ML-OL, SC และ SM ค่าแรงยึดเหนี่ยวของดินอยู่ระหว่าง 0.096 ถึง 1.196 กิโลกรัมต่อตารางเซนติเมตร และค่ามุมเสียดทานภายในของดินอยู่ระหว่าง 11.51 ถึง 35.78 องศาตามลำดับจากข้อมูลการสำรวจความต้านทานไฟฟ้า (สามแนวสำรวจ) ดินมีความลึกอยู่ในช่วงระหว่าง 2 ถึง 9 เมตร ความหนาของดินที่วางอยู่บนหินเป็นสิ่งสำคัญสำหรับการคำนวณปริมาตรของดินที่จะพังทลายลงมา ผลการวิจัยโดยสรุปพบว่า พื้นที่ที่เกิดการพิบัติของลาดดินมีความชันเท่ากับหรือมากกว่า 25 องศาโดยลักษณะของชั้นดินเกิดจากหินตะกอนเนื้อเม็ดที่ผุพังและมีความชื้นในดินสูง การเปลี่ยนแปลงค่าอัตราของความปลอดภัยของลาดดินจะเกิดขึ้นสูงที่ระดับความลึกของดินระหว่าง 2-5 เมตรซึ่งใกล้เคียงกับความลึกของการพิบัติในธรรมชาติ

ภาควิชา ธรณีวิทยา ลายมือชื่อนิสิต ____________________
สาขาวิชา โลกศาสตร์ ลายมือชื่อ อ.ที่ปรึกษาวิทยานิพนธ์หลัก ____________________
ปีการศึกษา 2555 ลายมือชื่อ อ.ที่ปรึกษาวิทยานิพนธ์ร่วม ____________________

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KEYWORDS : LANDSLIDE / UNDISTURBED SAMPLE / MULTI-STAGE DIRECT SHEAR TEST / SHEAR STRENGTH PARAMETER / RESISTIVITY SURVEY/ FACTOR OF SAFETY

THANAKRIT THONGKHAO : ENGINEERING AND PHYSICAL PROPERTIES OF RESIDUAL SOIL FROM LANDSLIDE HAZARD AREA IN AMPHOE CHIANG KLANG, CHANGWAT NAN, THAILAND. ADVISOR : ASSOC. PROF. MONTRI CHOOWONG, Ph.D., CO-ADVISOR : ASSOC. PROF. PUNYA CHARUSIRI, Ph.D., 197 pp.

Previous researches regarding landslide in Thailand have focused mainly to classify landslide hazard areas based on the analysis data in various factors such as topography, land-use, slope angle of soil slope, rainfall intensity etc. The field investigation and this research are aimed to geological setting, engineering properties of residual soil from parent rock including physical properties, properties of the soil shear strength decreases when the moisture content increases. In the study area, Ban Nong Pla, Amphoe Chiang Klang, Changwat Nan, found clastic sedimentary rocks are a parent rock including shale, sandstone, and siltstone. These residual soil and weathered rocks have a rate of high weathering therefore resulting to high susceptibility for landslide. As a result, the average value moisture content of soil, the average value specific gravity of soil, the average value liquid limit, and plastic limit are 24.83 %, 2.68, 44.93 % and 29.35 %, respectively. Plastic index is 15.58 % .Permeability value of soil is between as 9.36E-06 - 6.81E-07 cm/sec, respectively. Soils in the study area are classified by Unified Soil Classification System including ML-CL, CL- ML, ML- OL, SC and SM. The cohesion of soil values ranges from 0.096 to 1.196 ksc and angle of internal friction is between 11.51 - 35.78 degrees, respectively. Based on three resistivity lines survey, thickness of soil is between 2 - 9 m depth. The thickness of soil overlying on rock basement is important for calculating the volumetric of possible collapse. In conclusion, the result show that the failure slopes are greater than or equal 25 degrees, where soil profiles are overlying on the weathered clastic sedimentary rocks with high natural water contents. The change of factor of safety will occur where the soil depth between 2-5 m is located close to the depth of the actual slide zone.

Department : Geology Student's Signature ....

Field of Study : Earth Science Advisor's Signature ....

Academic Year : 2012 Co-advisor's Signature ....

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CHAPTER I

INTRODUCTION

Landslides are serious natural disasters in many parts of the world. UNESCO (2000) estimated that landslides claim about 1000 lives and 10-20 billion dollars of damages each year. Since the past 30 years, rainfall triggered landslides and debris flows had been one of the natural disasters of Thailand (Mairaing, 2006). Resettlement of the people in landslide risk area, deforestation, changes in land use and rainfall pattern are the factors that increase the potential damage in recent year. In Thailand the most well-known landslide event occurred in 22 November 1988 at Ban Kathun Nua, Pipun District, Nakhon Si Thammarat Province. As a result, approximately 230 were killed, 1,500 houses were damaged, and the total amount of 1,000 million bahts was estimated for the economic lost (Thantiwanit, 1992). In 2006, the disastrous landslide and flood in Uttradit, Sukhothai, Phare and Nan Provinces of northern Thailand made hundreds of people lost their lives and some became injured. Many parts of high mountainous areas in Thailand are exposed to landslide hazard occurrences.

1.1 Introduction

Landslide is one of the natural disasters cause damage to life and casualties in Thailand every year. A number of factors trigger the landslide commonly include heavy rainfall during tropical monsoon, geological condition, and change of land uses .The direct economic lost due to landslide is calculated to be equal to 100 million Baht per year and the return period of large area landslide is once in every 3-5 years (Soralump, 2010). Factors considered include 1) landform (slope and elevation), 2) geologic condition (rock type and lineament zone), 3) land-use, 4) distance from surface water, 5) soil characteristics, 6) rainfall intensity and 7) geotechnical engineering properties of residual soil. The assumptions for the cause of increasing number of landslide for the past decade are 1) landslides naturally occur more often which might be related to the climate change, 2) mismanagement of land use due to the increasing number of population and the needs of land for producing agricultural products, that force people to stay in the landslide hazard areas and 3) combination of first and second reason.

Table 1.1. Number of casualties from landslide events in Thailand since 1970 (Soralump, 2007).

Area	1970 - 2006		Average lost (37 years)	
	Lives	Economics (Bath)	Lives	Economics (Bath)
North	286	2,575,600,000.00	8	69,610,810.81
Central	1	300,000,000.00	0	8,108,108.11
South	247	1,010,000,000.00	7	27,297,297.30
Total	534	3,885,600,000.00	14	105,016,216.22

It is very important to understand the mechanics behavior of residual soil or weathered rock in order to understand the landslide behavior. Therefore, the classification of rock group that specifically related to landslide is needed at the first place. Rainfall intensity and geotechnical engineering properties factor is important to analysis the landslides hazard area (Soralump, 2010). In addition, the large area landslide is natural and mainly caused by unusual large precipitation in the area. However, there are also many evidences that deforestation or agricultural process is the main cause of large area landslide. Rainfall-triggered landslide has been considered one of the major natural disasters in the last decade. It becomes the severe problem in the mountainous area of many countries includes Thailand. It causes large damage and casualties every year. The economic loss from this hazard is beyond the expectation.

On the natural slope, behaviors of unsaturated soil and its shear strength are very sensitive to the water content change caused by rainfall. The water content increasing cause decreasing of soil suction in consequence the shear strength is decreased (Mairaing, 2006). During the rainy season, the soil is at the marginal stable state because of the high humidity. Additional loss of shear strength from the rain can easily causes landslide. As a result, rainfall data is important for the warning system. However, in Thailand, the development of the warning system cannot be done easily because of lacks of landslide records and rainfall data gathering. The goal of study to the landslide hazard susceptibility analysis is to assess the landslide hazard area in order to reduce the risk to people, urban area, road, building and plantation area. In addition, Rainfall - triggered landslide is proved to be manageable if warning criteria and hazard area can be supplied from research field. In turn, in order to obtain those data for large area, comprehensive geotechnical testing shall be done. Area should be grouped based on geological group, engineering properties of geo-material of each group must be determined (Soralump, 2010). Strength reduction model and infiltration model are important tools to calculate the amount of rainfall that may trigger landslide. The final goal for landslide analysis is the given the proper warning to the public concerned. In order to do so, we can establish the warning levels from the simple index such as the accumulative rainfall from the field or with the Geographic Information System technique, the semi-real time calculation of Factor of Safety. However, the prediction model has to be calibrated with the known landslide even with some field monitoring instruments as mentioned above.

In August 23, 2006, a landslide was initiated in many areas of Nan province after five days of the unusual heavy rainfalls. This event caused many houses, buildings and bridges completely destroyed (Akkrawintawong, 2008). Landslide locations or scars were identified in the Nan province from interpretation of aerial photographs as well as enhanced Landsat 7 ETM and IKONOS image for constructing the scar map. As a result, that the open-forest areas dominated by the steep slopes, sandstone, granite and tuffaceous rocks, proximal long faults/fracture, are prone to landslide hazard. Subsequently, degree of landslide hazards were expressed as very low to very high hazard levels. The landslide hazard map also indicates that high- to very high- level hazard areas is the mountainous areas in Chalerm Prakiat, Song Khwae, Chiang Klang, Thung Chang, Pua Districts. Several districts, such as Na Noi, Wiang Sa and Nan in the southern part of the Nan province have much lower hazard level. Therefore, Ban Nong Pla area, Amphoe Chiang Klang, Changwat Nan, Thailand was selected for landslide hazard investigation and was selected for the case of this study.

This research will focus on analyze engineering properties and physical properties of residual soil from a parent rock and collected residual soil from landslide hazard area for analyze the shear strength parameters. The relationship between shear strength and moisture content behavior of residual soils will be discussed. The factors used to analyze the stability of soil slopes are also analyzed in order to find the plane of failure and find the safety factor of the soil slope.

1.2 Objectives of Thesis

1) To classify patterns of landslides in Ban Nong Pla Village , Amphoe Chiang Klang, Changwat Nan from remote sensing data.

2) To analyze engineering properties and physical properties of residual soils from a parent rock collected from landslide hazard area; and

3) To explain the relationship between shear strength and moisture behavior of residual soils in Ban Nong Pla Village, Amphoe Chiang Klang, Changwat Nan.

1.3 Scope and Limitation

This research employs the remote sensing and geographic information system techniques for the specific mapping (for example, lithological, slope angle, slope direction, soil group, land use, drainage basin, elevation, topography) and classification different between landslide scars and vegetation area in the study area. In addition, classifying the patterns of landslide from the physical factors data in the study area is also carried out. The landslide susceptibility area was derived by incorporating physical factors and analyzed using weighting factors. The research focuses on the explain relationship between shear strength and moisture content behavior of residual soils collected from landslide hazard area. In addition, prediction and warning by

Geotechnical Engineering Method is the direct method for calculating the Factor of Safety (F.S.) of soil slope during heavy rainfall.

1.4 Expected Results

1) Understanding the patterns of landslide from the physical factors data.

2) To analysis of the slope stability and prediction model factor of safety the soil slope.

3) The relationship between shear strength and moisture behavior of residual soils from Ban Nong Pla Village, Amphoe Chiang Klang, Changwat Nan.

1.5 Characterization of the study area

1.5.1 Topography

Ban Nong Pla village is approximately 4 square kilometers in area and located in the northeastern part of Amphoe Chiang Klang, Changwat Nan, Thailand (shown in Figure 1.1). Ban Nong Pla Village is located on the valley with steep slope and high elevation. In addition, the study area is surrounded by mountains with high of 700 -1200 meters above mean sea level. Direction of flow within the sub watershed in the area is from southeastern to northwestern parts into the Huai Nam Puea catchment. Populations mostly in the area are farmers and agriculture (for example, rice plantation, lychee plantation, corn for animal feed plantation). The development tends to move higher to the mountain and change of the land use and land cover caused the landslide in the area such as shown in Figures 1.2 and 1.3. The consequences of population growth and demand for agricultural land cause the deforestation alter the land -use, human activity has resulted in the progressive erosion. Furthermore, in the rainy season starts at May to October in Nan province. A landslide has happened in a high mountainous area and soil slopes that have been made in the road after heavy rainfalls. Thailand is located in the warm and tropical climate region. The tropical monsoons and typhoons from both the Andaman Sea and the South China Sea contribute to the heavy rain in the region. Rainy season starts in June commonly produces the over discharge in the basin locating in the northern part of the country. Not only the effect of heavy rain in the northern region, long and continue heavy rain in the southern Thailand ending season in December also cause the overbank full discharge in many places. The average annual rainfalls are ranging from 1000 to 1500 mm. for northern, northeastern and central parts of the country. At the eastern part and southern peninsula, the rainfalls are averaged from 2000 to 3000 mm. The rainfall induced landslides are normally occurred on the mountainous area due to single intent or long period rain. In case of prolong rainfall, the flood and debris flow will follow the landslides and cause more damage to the villages along flow passages and on the alluvial plain below.

The failure of slopes in the steep terrain is often occurs mostly in conjunction with flooding. Cause of the failure slope includes natural factor such as topography. Slope factor and the soil factor and the other causes include human activities such as changes in soil slope to make agriculture and changes in land use.

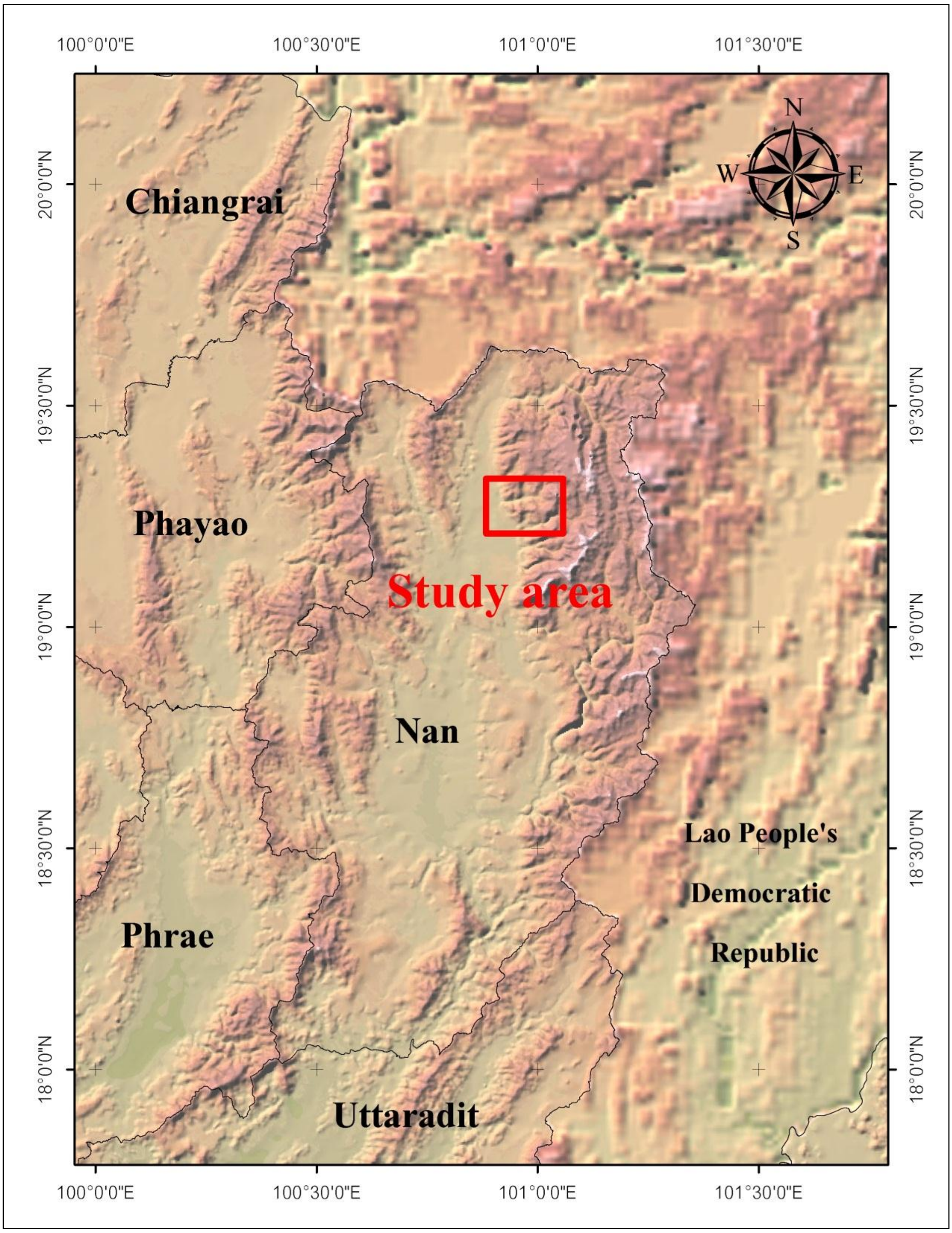

Figure 1.1. Ban Nong Pla Village located on the northeastern part of Amphoe Chiang Klang, Changwat Nan.

Figure 1.2. Ban Nong Pla Village located on the valley with steep slope and high elevation.

Figure 1.3. Landslide occurrence (cut slope failure) in the Ban Nong Pla area.

1.5.2 Geological Condition

Geology is also the main factors influent landslide. From literatures, granite and granitic soils show the highest potential of landslide, whereas some sedimentary rock groups such as shale, mudstone, and siltstone influencing by geologic structures such as the bedding planes, joints, faults are second highest group (GERD, 2006) as shown in Table 1.2.

Table 1.2. Risk of geology for landslide (GERD, 2006).

Level	Landslides potential	Rock Group	Geologic structure influence
1	Highest	**Granite Dominated:** Highly and deep weathering zone, thick residual soil.	low
2	High	**Shale and Mudstone Dominated:** High weathering rate, shallow residual soil.	High
3	Moderate	**Sandstone and Siltstone Dominated:** Moderate weathering rate, shallow residual soil.	High
4	Low	**Quartzite, Sandstone, Siltstone Dominated:** Similar to Level 3 but quartzite is stronger to The weathering processes.	Moderate
5	Lowest	**Limestone and Dolomite rocks:** moderately weathering rate, shallow residual soil.	Low

Geological characteristic of Nan province is identified into 10 units of the following types of rocks (Land Development Department, 2006).

1.1 Unit 1 sand sediment, clay, fine gravel

This unit consists of clay, gravel, sand and gravel. The influence of slope and surface water are mixed, the various types of sediments are present. The terrain is flat along the river. Some sediment can be a source of construction sand and clay as raw material for the pottery industry. The soil is rich in minerals needed for plants and suitable for growing. However, due to the flood plains are often faced with the threat of flooding during the rainy season on a regular basis. The sediment was found in the plains along the major river basin that is the Nan River. The formation of large sediment found from the northern, central and southern districts in Thung Chang, Chiang Klang, Pua, Tha Wang Pha, Muang, Phu Phiang, Wiang Sa, Na Noi districts

1.2 Unit 2 Gravel sediment, sand, gravel

Unit 2 consists of silt and gravel and relatively thick with alternating layers of sand and clay. Rounded pebbles look great in size from 2 mm to 1 m larger than some of the iron oxide solution interface into the gravel. This unit forms as the stair-step topography due to erosion of the surface of the river. The soil is fertile enough to grow some plants. This area is not located in a flood zone so that there is suitable for housing, but may face a variety of water flow. The sediment was found to be sand, gravel, clay

1.3 Unit 3 sedimentary rocks: types of siltstone, coal, shale

Unit 3 consists of clay, siltstone, ball clay, coal and oil shale and semi-solid. The fossil species of freshwater, bivalves, fish, mammals and more. Fossils can be found in similar species to Krabi province. An accumulation of mineral fuels such as oil, natural gas, coal and oil shale are also found in soil, light and with ball clay. This type of rock forms and accumulates in sedimentary basins between mountains and freshwater lake environments. The rock of unit 4 was found at the north of the province, including the Thung Chang basin, Chiang Klang basin, Pua basin, Tha Wang Pha basin. They also found in a narrow space in the central and southern provinces, including pools and Wiang Sa, Na Noi basin, respectively.

1.4 Unit 4 sedimentary rocks: types of sandstone

Unit 4 is composed of sandstone such as quartz sandstone texture, grained sandstone and volcanic ash, feldspar, and the sandstone texture. They are also discovered as the small gravel, sandstone, shale, chert, volcanic ash and limestone, a sedimentary-shirt to a certain range. Grained sandstone in the area was used as the stone and the stone knife. The soil is rich in minerals, except for plants of medium grained sandstone with quartz minerals which are relatively low. The stone was found near the mountains and distributed as spread over an area of the province, including Doi Sa Pharm Maw, Doi ma Hae in Song Khwae district, Doi Phu wee mountain, Doi Phi Kip Kae, Doi Mon Phe Tay, Doi Khun Lan and Doi Hway in Charoem Prakiat, Bookie district and Mae Charim district, Doi Khun Mae Gard, Doi Nong Luang, Doi Mae Jok in Ban Luang district, Wiang Sa and Na Noi districts, Doi Pha Ki, Doi Pu Fah, Doi Dong Khway in Ban Luang district, Muang and Wiang Sa districts, Doi Pha Ngam, Doi Mho Tom in Wiang Sa and Na Noi districts.

1.5 Unit 5 sedimentary rocks: types of shale

Unit 5 consists of shale, chert, siltstone, sandstone, limestone and volcanic ash. The area remains in a mountainous area with a high risk of further landslides. The weathering soil was weathered from the shale that contains mineral-rich medium. The soil may be less incoherent. The rock unit was found in the area along the north - south of the province including Doi Khun Nam Gorn, Doi Khun Satun in Charoem Prakiat district, Chiang Klang district and Pua district, Doi Pha Luang, Doi Khun Wow, Doi Phae Kwang, Doi Khon Kaen in Song Khwae district, Tha Wang Pha district, Phu Phiang district and Wiang Sa district, Doi Sam Sob, Doi Nam Aun, Doi Su Tho in Na Noi and Na Mun district. This unit is also found in a narrow mountain such as Doi Toon at the western province in Ban Luang district.

1.6 Unit 6 sedimentary rocks: type of limestone

Unit 6 consists of dark gray limestone and occasionally found shale, sandstone and limestone with dolomite texture. The terrain is steep and there are many great causes at the beautiful landscape. Limestone is used as raw material in the chemical industry and also been used as a building material as well. Limestone is soluble in water with a mild acidity. Therefore, it is often found in caves with stalagmites and stalactites in a limestone mountain. Although the steep limestone cliffs and clear. Therefore, it is not an area prone to landslides, but the phenomenon may well collapse in the plains near the mountain limestone. The weathering soil was from the limestone that shows an orange and red color, is called “Terra Rosa”. Especially iron, calcium and magnesium are favored for plants. The limestone plateau distributes in the northern areas of the province including Ban Huai dong in Charoem Prakiat district, Ban Khun Nam Hae in Thung Chang district, Doi Hin Khwae and Doi Kra thing in Chiang klang district, Doi Jee, Ban Pha Lang, Ban Nam Koa Noi in Song Kwae district, Doi tum Pha Kao, Ban Nam mow, Ban Yod Doi Patana in Bokuai district and Ban Mai at Ban Luang district.

1.7 Unit 7 metamorphic rocks: types of slate, schist and quartzite.

Unit 7 includes low-grade metamorphic rocks of slate, schist and quartzite. Some areas remains high mountainous areas prone to landslides have also occurred. The weathering of the phyllite, slate and schist can take advantage of the medium and growing of landslide quite well. The quartzite is strong and resistant to weathering, high, therefore, it does not cause landslide problem. The weathering soil was from these rocks that minerals in the soil are relatively low. The rock unit 7 contains joint and fractures and found at the Nan province to Uttaradit province and found in Doi Tong yang, Ban Pak lee at Na Mun district.

1.8 Unit 8 Volcanic rock: types of Rhyolite, Andesite

Unit 8 consists of rhyolite, andesite, basalt, volcanic ash. The weathering of volcanic rock was easily destroyed. The area near the high mountains of volcanic rocks is high landslide risk area. It is very suitable for agriculture. The rock unit 8 was found in the mountains at the western part of the province, including Ban Na Ka Phu Nang Klam in Ban Luang district. and Wiang Sa district. Doi Phu Keng, Doi Phu Fah , Doi Phu Lerm in Muang district and Wiang Sa district, Doi jaunt Phra Sard in Na Noi district and Doi Khun Mae Kard in Bokuai district.

1.9 Unit 9 Intrusive igneous rocks: type of Granite, Diorite

Unit 9 consists of granite, diorite and the terrain is high mountainous area. Granite has a close relationship with the origin of the mineral economy of various kinds, such as tin, tungsten, fluoride and other barite. The rock unit 9 was found in the northeastern part of the province, including Ban don Phu Kha, Ban Yod Doi Patana in Tha Wang Pha district and Bokuai district and the south area of the province including the Ban Huai lord in Wiang SA district and Doi Luang in Mae Charim district.

1.10 Units 10 Ultramafic igneous rocks: type of basalt, pyroxenite

Unit 10 consists of basalt, pyroxenite, peridotite and metamorphic rocks. Serpentines are black, dark green corrosion. The terrain in this area is high mountain, but these areas have a bundle of metal ores such as nickel, cobalt, magnesium and copper species. In addition, the rocks are commonly strong eroded. Basalt is employed for building and substitute for limestone. The weathering soil was from the basalt rock that has high abundance of rock suitable for agriculture more generally. The rock unit 10 was found in two areas including Ban Nam pang nun, Ban Pae in Mae Charim district and Doi Puk Jum Peng, Doi Khum Kao, Doi Jum bun in Na Noi district.

1.5.3 Climate and rainfall

Thailand is located on the warm and tropical climate region. The tropical monsoons and typhoons from both Andaman Sea and South China Sea contribute to the heavy rain in the region. Rainy season starts at June on the northern part and ends at December on the southern part of the country. From the climate information of Nan Province for a mean of 30 years (1975 - 2004), this is in the area of tropical rain or tropical grassland. This area is commonly face with the season of rain and will be the influence of two types of monsoon winds from the southwest monsoon in China begins to blow in from May to October the rainfall in this period. The northeast monsoon begins to blow from November to January will blow cold cover and arid northern region of Thailand. The weather at the Chiang Klang district, Nan province can be divided into 3 seasons, including summer season (March – May), rainy season (June – October) when the study area is affected by southwestern monsoon, and winter season (November – February), when the study area is influenced by the northeastern monsoon blowing the coldness from the North Pole and China – Siberia to Thailand. In addition, rain begins to fall moderately in April, the average 9 days/month, and rainfall from May to October average rainfall is 10 to 23 days/month for a rainy day, the highest August average 23 days/month. The rain was the least in January. The average rainfall in December and a day/month and average annual rainfall are 119 mm.

The average annual rainfall of 1,255.60 mm can be measured at the Meteorological station of Nan province during year 1998 - 2004. The average maximum rainfall of 265.4 mm is record in August, and the average minimum rainfall of 8.1 mm can be measured in December. The average temperature of 28º C can be measured during 1998 -2004. The average highest temperature of 32 º C can be measured in May and the average lowest temperature of 21º C is recorded in January. The average relative humidity of 78 Percent is measured during year 1998 – 2004. The average highest relative humidity of 86 percent can is record in August. The average lowest relative humidity of 65 percent can be measured in March and the average annual relative humidity is 77 percent.

1.5.4 Land use

Landslide problems in Thailand were caused by the combined effects of the natural and manmade factors. In the past, when the areas are still in untamed forest, landslide usually occurred after forest fire or extremely draught year. In the past, when the areas are still in untamed forest, landslide usually occurred after forest fire or extremely draught year. The evidences of large alluvial fans from landslides can be noticed on the west of central plain at Kanchanaburi (Mairaing, 2006). Land use is one of the factors contributing to landslide. The consequences of population growth and demand for agricultural land cause the deforestation alter the land use. Many human activities such as opening areas for hunting by fire or eradication of native vegetation for farming have enhanced the problem of soil erosion.

In addition to that, the evolution of agriculture left an ever-stronger imprint on the land in many regions. Sureeratna L. (2001) reports that Thailand's forest areas declined from 53.33 percent of the total land in 1961 to 25.02 percent in 1999 as shown in Table 1.3.The changing of rainfall pattern, infiltration rate and flow regimes within soil profile due to cultivation can also trigger the landslides.

Land use mostly in the study area was farmers and agriculture, for example, rice plantation, lychee plantation, corn for animal feed plantation as shown in figure 1.4. The people mostly in the village have destruction of natural forests for using in the cultivated area. It was found that more than 95% of limited area landslides are always caused by the disturbance of human activities such as highway slopes, building and agricultural development on hillside which changes the landform or surface and underground water flow characteristics. When the large scale landslides are usually occur on natural slopes due to heavy rain. However, there are also many evidences that deforestation or agricultural process is the main cause of large area landslide.

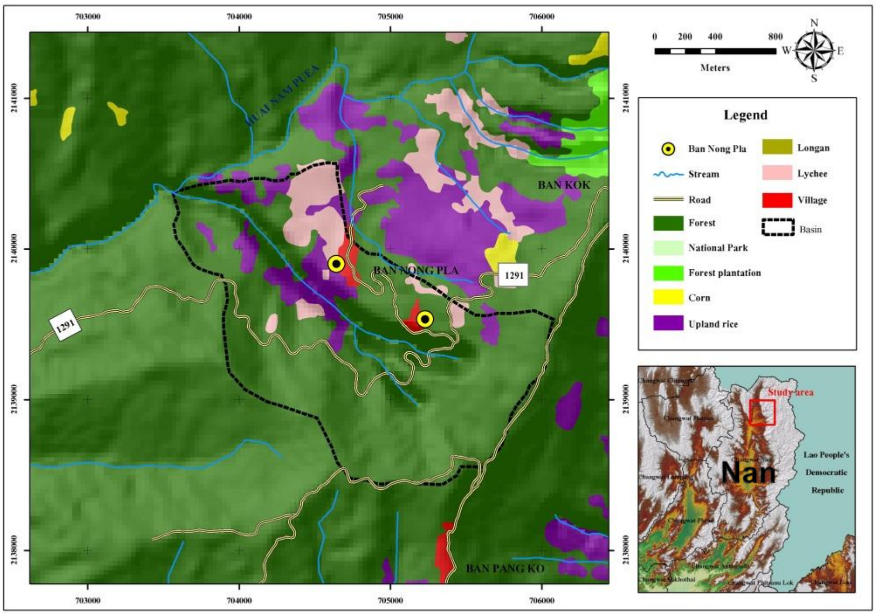

Figure 1.4. Land use of Ban Nong Pla Village (Land development department, 2009).

Table 1.3. Status of forest area in Thailand: 1 hectare (ha) equals 6.25 rai (Thai measure) (Sureeratna, 2001).

Year	Remaining Forest (rai)	Remaining Forest (%)
1961	171,017,812	53.33
1973	138,578,125	43.21
1975	128,278,755	40.00
1976	124,010,625	38.67
1978	109,515,000	34.15
1982	97,875,000	30.52
1985	94,291,349	29.40
1988	89,877,182	28.03
1989	89,635,625	27.95
1991	85,436,284	26.64
1993	83,470,967	26.03
1995	82,178,161	25.62
1998	81,076,428	25.28
1999	80,242,572	25.02
	12,838,811.6 ha	

1.5.5 Geomorphology and parent materials of soil

Nan province area is flat and covered with sedimentary rocks of Korat Group which consists of sandstone, siltstone, shale and conglomerate. These areas are displayed as high mountainous side, especially in the west of the province and the content of the various rock layers, while the eastern provinces displays as the depositional basin.

The geomorphology (land form) of the Nan province can be classified into groups as follows.

1. Flood plain (flood plain and recent river alluvium) is mostly flat. This group is also characterized by natural levee and the depression behind the ridge of soil water (back swamp) on the sand ridge and oxbow lake.

2. River terrace is relatively new (low terrace of semi-recent river alluvium) consisting of clay, loamy and clay silt and clay.

3. Old terrace (middle and high terrace of old river alluvium) consists of sandy loam soil.

4. The alluvial fan in the high (High closing fans of old colluvium) deposition is characterized by clay loam soil mixed with gravel.

5. The residual from erosion (erosion surface and residual hills) contains several soil types, depending on the type of soil parent materials.

6. Slopes different level (hill land).

7. Mountainous regions (mountains).

1.5.6 Soil resources

The 17 soil groups were classified from Nan province that covers an area of about 5,991,792 acres or 83.57 percent of the area (Land Development Department, 2011) as shown in Table 1.4. The entire map is divided into 37 units or 30 units of water and soil mixed units 1 and 6 were mixed in each map unit, which represents the type of slope such as.

5 : 5 soil group 0-2 percent slope.

46B : 46 soil group 2 - 5 percent slope.

29B / b : 29 soil groups were adjusted for area farmers.

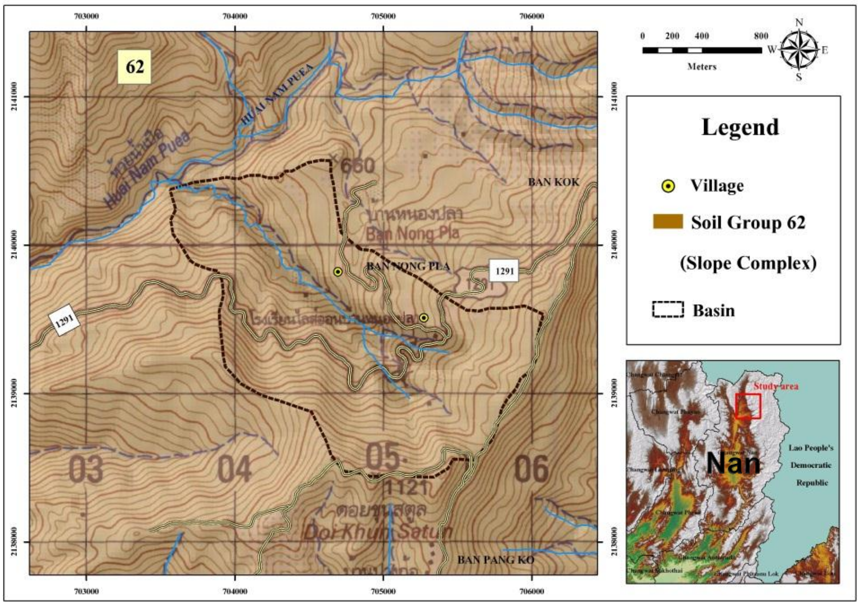

Figure 1.5. Characteristics of soil group in Ban Nong Pla village (Land development department, 2009).

Characteristics and properties of soils found in the Nan province can be described as follows.

1) Soil group 5

A clay soil is very deep gray. The soil parent materials are river sediments. There is a wetland area or a flat. Some areas are found in low lying areas. Flooding and high water runoff in the rainy season or the plaster tablet is 100 cm in depth from the surface. The soil layer is very deep, loamy and clay, gray with a dash of yellow or brown. Soil reaction is moderately acid to slightly acid with a pH of approximately 6.0 to 6.5 with the lower soil is clay or silty clay last year. The reaction is slightly acid to alkaline medium. Value of about pH 6.5 to 8.0 is found before they accumulate as iron and manganese in the soil. The soil was found such as. Hang Dong series (Hd) and Phan series (Ph).

2) Soil group 46

The terrain is undulating to hilly with good drainage, moderate to good. The top soil is loamy and clay. Soil reaction is extremely acid to slightly acid. With a pH of approximately 5.0 to 6.5 with clay or clay loam soil layer below the gravel is present within a depth of 50 cm from the surface to brown or reddish brown. Soil reaction is extremely acid to slightly acid. Value of about 5.0 to 6.5 pH with a thick layer of gravel or rock fragments is greater than 100 cm of the soil surface. Some crops include such as cassava, sugarcane, jute, and some of the natural grassland and scrub or reforestation .Issues of land use. The natural abundance of low water areas that is prone to erosion and soil erosion. The soil was found such as Chaing Kan series (Ch).

3.) Soil group 29.

A clay soil is very deep. The terrain is relatively flat to rolling hills. Good drainage. Some areas may be found in the gravel bottom. The soil is a loamy clay soil, brown, yellow or red up to acid soil reaction with a pH of approximately 4.5 to 5.5 with ground beef, brown clay floor. Some areas remain as natural forest. Areas are prone to high erosion soil erosion and water scarcity. The soil was found such as Baan Jong series (Bg) Chaing Khong series (Cg) and Mae Tang series (Mt).

Table 1.4. Area of the soil, other areas and soil series in the Nan province (Land Development Department, 2011).

order	Soil group / Other areas	Soil series	Area(rai)	percentage
1	5	Hd-siclA, Ph-siclA	10,479	0.15
2	7	Na-siclA, Skt-siclA	44,405	0.62
3	15	Mta-silA	84,466	1.18
4	29B	Bg-clB, Mt-clA	58,554	0.82
5	29B/b	Mt-clA/b	4,004	0.06
6	29C	Bg-clC, Cg-clC	24,702	0.34
7	31B	Wi-cB	2,358	0.03
8	31C	Wi-clC	1,903	0.03
9	33/38B	Tph-silA/Cm-silA	39,057	0.54
10	33B	Tph-silA, Kp-silA	4,456	0.06
11	35B	Hc-slB	40,907	0.57
12	35C	Hc-slC	3,915	0.05
13	40B/b	Sp-slB/b	1,501	0.02
14	46B	Ch-clB	3,980	0.06
15	46C	Ch-clC	34,618	0.48
16	47C	Li-clC, Ml-clC	101,485	1.42
17	47D	Li-clD, Ml-clD, Tl-clD	130,620	1.82
18	47E	Li-clE, Ml-clE, Tl-clE	17,405	0.24
19	48B	Mr-slB, Pao-slB	15,834	0.22
20	48C	Mr-slC, Pao-slC	36,330	0.51
21	48D	Mr-slD, Pao-slD	31,824	0.44
22	55B	Ws-clB	5,222	0.07
23	55C	Ws-clC	17,114	0.24
24	55D	Ws-clD	1,261	0.02
25	55E	Ws-cl\E	839	0.01

26	56B	Png-slB	1,810	0.03
27	56C	Png-slC	6,997	0.10
28	59	AC-pd-silA	61,724	0.86
29	59B	AC-pd-silB	796	0.01
30	60	AC- wd-slA	9,732	0.14
31	62	SC	5,193,494	72.43
32	Land converted (ML) such as Hospital, Industrial factory, Housing estate, Golf Course		3,491	0.05
33	Space filled with rocks (RL)		1,032	0.01
34	Urban areas (U) such as Housing, School, Temple		86,748	1.21
35	Wildlife Sanctuary (WS)		60,202	0.84
36	Water (W)		18,142	0.25
37	National Forest area (NP)		1,008,638	14.07
Total area			**7,170,045**	**100.00**

1.6 A brief guide to the Thesis

This thesis provides a landslide data of the Ban Nong Pla Village, Amphoe Chiang Klang, Changwat Nan. Thesis is divided into 6 Chapters and detail of each chapter is as follows.

Chapter I - Introduction, objectives of thesis, scope and limitation, expected results, characterization of the study area and nearby.

Chapter II - Literature reviews, definition of landslide, types of landslide factors controlling landslide, including previous works relevant researches of landslide and case study of landslide in northern Thailand.

Chapter III - Methodology, data collection associated with landslides, including part of using GIS and remote sensing interpretation, explain the soil sampling in the field and 2-D electrical resistivity survey, physical and mechanical properties of soils, soil classification, and slope stability analysis.

Chapter IV - Results, including of geological setting and geological structure in the study area, interpretation of aerial photograph or satellite image from remote sensing data, 2-D resistivity interpretation (study soil layer overlaying on bed rock), including part of physical properties and engineering properties of residual soils collected from landslide hazard area, field soil classification (USCS), safety factor of soil slope.

Chapter V - Discussion, with special emphasis on the explain relationships between shear strength and moisture content behavior of residual soils, shear strength of soil decreases when the moisture content increases , Relationship between factor of safety decreases when slope angle increases ,The change of factor of safety decreases when thickness of soil increases. Based on resistivity lines survey, the thickness of soil overlying on rock basement is important for calculating the volumetric of possible collapse.

Chapter VI - Conclusion and recommendation of results gathered from the study research.

CHAPTER II

LITERATURE REVIEWS

2.1 Introduction

The chapter II presents the literature reviews all about the definition of landslide, types of landslide, relationship between landslide and water, landslides and seismic activity. Factors controlling landslide in relevant to the previous researches in the northern Thailand were also reviewed. Landslides and other slope movements have attracted the attention from scientific studies. As previous researches suggested that the problem of the stability of slopes both occurred naturally and/or by the excavation, has challenged many research fields of particularly geology and civil engineering. In terms of geology, the difference of rock types where the top beds are separated from the underlying stationary part of the slope by a definite plane of separation, rapid movements of sliding rocks can be occurred.

Deformations of slope in a long term are also considered sliding phenomena. They do not occur in the surficial zone but occur within a thick zone consisting with a system of partial sliding planes (Cruden, 1991). These deformations possess the character of a viscous movement and are termed the “creep”. The best results of landslide studies can be achieved only by the combination of both geology and engineer approaches. The quantitative determination of the stability of slopes by the methods of soil mechanics must be based on geological structure of the area, the detailed com position and orientation of strata and the geomorphological history of the land surface. In addition, geoscientists may obtain a clearer picture of the origin and character of sliding process by checking their considerations against the results of static analyses and the research done by means of engineering properties of soil and rock mechanics.

A good example of this is the approach to the classification of soil material involved in landslides. Some geological engineering classifications divide soils into two groups, earth in which 80 percent or more of the particles are smaller than gravel (2mm diameter) and debris-with more than 20% of gravel and coarser (Varnes, 1978:, Cruden and Varnes, 1996).

For geotechnical engineers, the primary division is between cohesive and cohesionless soils, which have fundamentally different mechanical and drainage characteristics (Morgenstern, 1992). There is almost no relationship between these two fundamental groupings. Poorly sorted debris with as much as 65% coarse grain content by volume can be cohesive, when supported by a clayey matrix (Rodine and Johnson, 1976). Laboratory tests by Holtz and Ellis (1968) showed that adding up to 35% by weight of gravel to a well-graded clayey soil resulted in no measurable change of the effective friction angle and cohesion. Therefore, the limiting 20% content of coarse clasts, used in the engineering geological classifications, has little significant with respect to the mechanical behavior of the soil. Similar discrepancies exist in the understanding of other factors such as failure mechanisms, saturation and pore-pressure changes by the two groups. The following discussion of landslide classification and terminology attempts a balanced view, based on both geological and engineering outlook.

2.2 Definition of landslide

A standard set of terms for description of landslides has been suggested by the Commission on Landslides and Other Mass Movements of the International Association for Engineering Geology and the Environment (IAEG 1990) and the UNESCO Working Party on the World Landslide Inventory (WP/WL1). This includes a general definition of a landslide, which contrasts with an earlier perception that the word “landslide” specifically implies sliding movement (Cruden, 1991):

Landslide is “the movement of a mass of rock, debris, or earth (soil) down a slope (under the influence of gravity)”.

The standards also define a large number of useful terms describing the dimensions and morphology of landslides. Many of these are summarized in Figure 2.1. These terms appear to have been equally well accepted by both geologists and engineers. The “rupture surface” is often called “sliding surface” in engineering literature, reflecting a preoccupation with sliding failure mechanisms.

Some of the morphological terms are difficult to apply to landslides with long run out such as debris flows or debris avalanches, where the dis placed mass completely vacates the space above the rupture surface and moves down the slope (Mencl and Záruba ,1969). The space between the rupture surface and the original ground could then be called “source”. This is a useful addition to the “zone of depletion” for all landslides. The strip of land traversed by the moving mass down-slope from the source may be referred to as “path” or “trail”, while the “accumulation zone” in this case coincides with the “deposit” as shown in Figure 2.1 (King, 1999). The word “run out”, can be used to describe the distance between the toe of the source and the toe of the deposit.

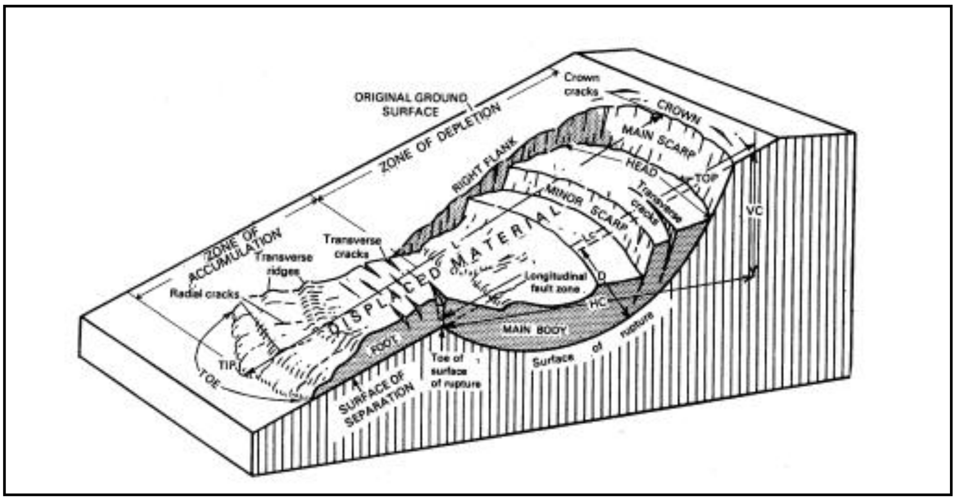

Figure 2.1. Terms describing the morphology of a landslide (Cruden and Varnes, 1996).

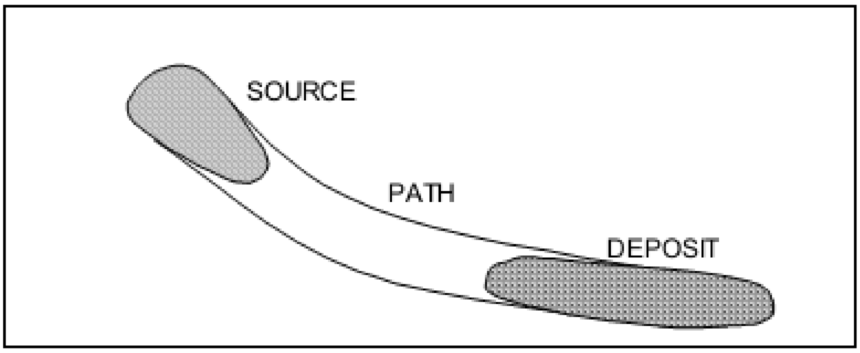

Figure 2.2. Component of a long - run out landslide concept (King, 1996).

The term “travel distance” is defined to describe the distance from the crown of the rupture surface to the toe of the deposit (L in Figure 2.3). The travel distance angle, Φ_a, is defined and shown in Figure 2.3.

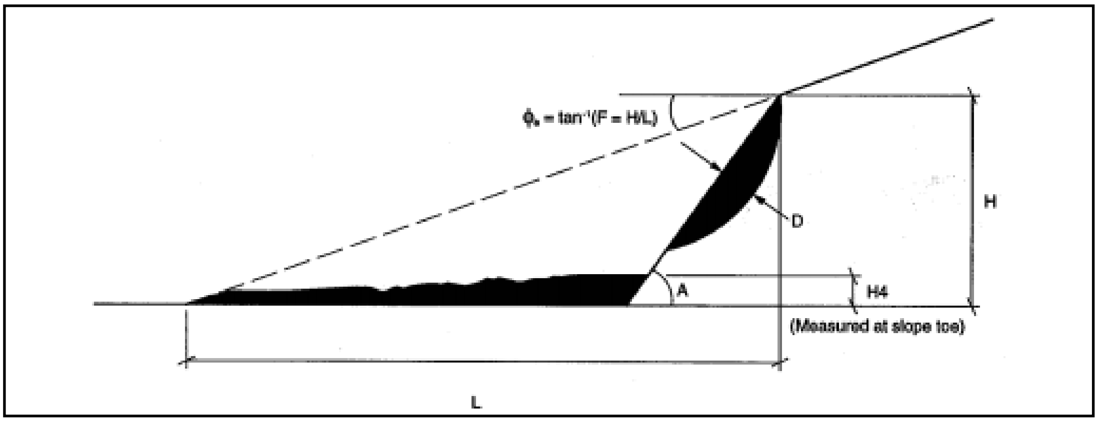

Figure 2.3. Component of a long - run out landslide concept (King, 1996).

Criteria for classifying of landslides and the erosion of the hillside are based on several mechanisms such as speed and movement of sediments, trace the shape of the landslide and the amount of water involved in the landslide. Types of landslides include the widely used classification by (Varnes, 1975) which relied on the principle of classification of the avalanche of material (type of material) and the movement (type of movement).

2.3 Types of landslide

Landslide is a world - wide natural hazard, which in most cases occur as a debris flow and is caused by intense and continuous heavy rainfall (Cruden and Varnes, 1996). Downward and upward movements of the slope-forming materials, which can be rock, soil, artificial fill, or a combination of these materials, can lead to many kinds of mass movement such as falling, toppling, sliding, spreading, or flowing. The commonly accepted terms describing landslides are show in Figure 2.4. Apart from mountainous regions, the landslide can also low relief areas as cut-and-fill failures (roadway and building excavations), river bluff failures, lateral spreading landslides, collapse of mine-waste piles (especially coal) (Cruden and Varnes, 1996).The most common types of landslides are described as follows and are illustrated in figure 2.4.

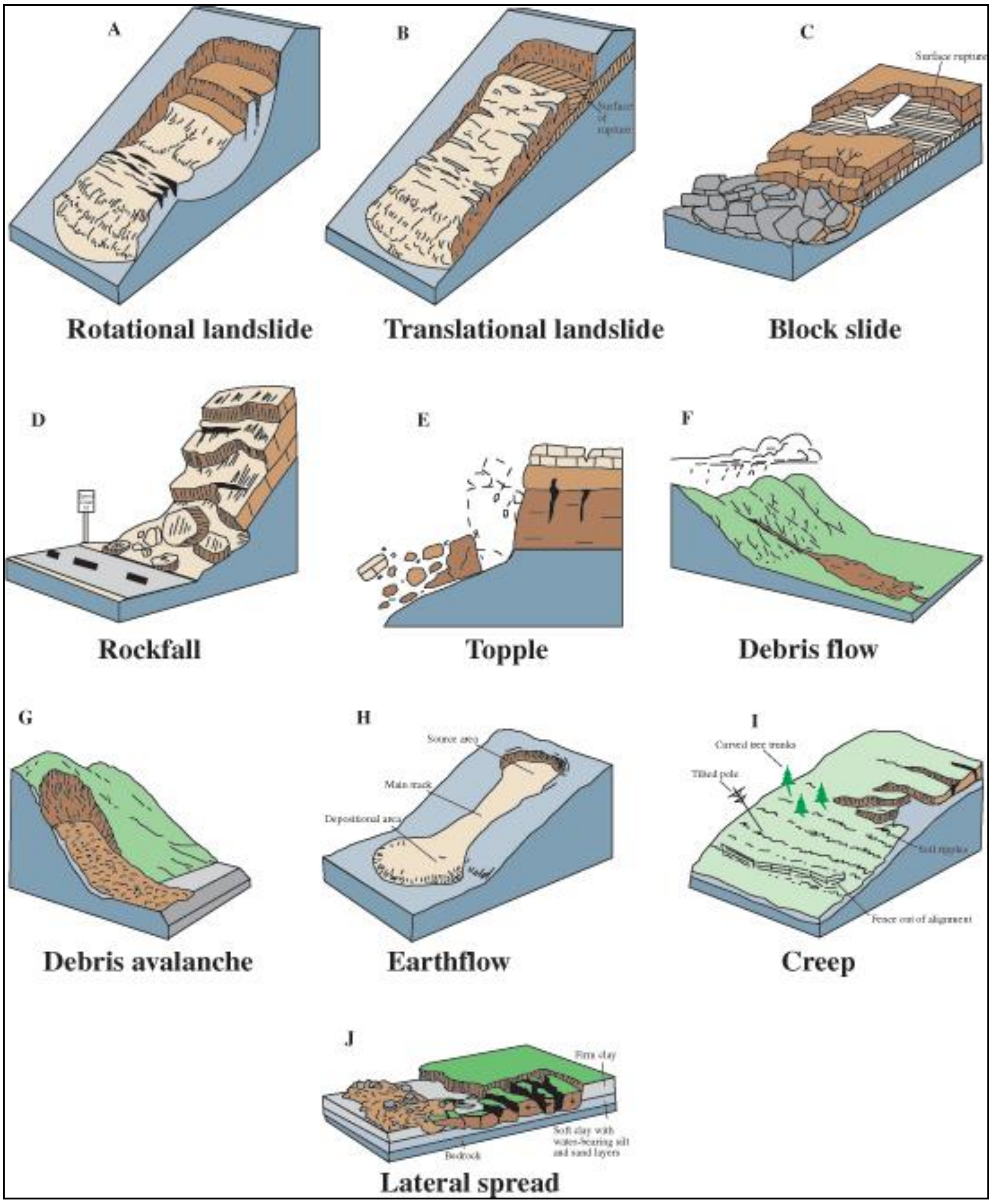

Figure 2.4. Terms describing the type the landslide (http://pub.usgs.gov/fs/2004/3072 /image/Fig3grouping-2LG.jpg)

2.3.1 Slides

Even though many types of mass movement are general terms of “landslide,” the more restrictive mean of this term is the mass movement in a distinct weakness zone over more stable underlying material (Cruden and Varnes, 1996). The two major types of slides are the slides called “rotational slide” and “translational slide”. The rotation slide refers to the slide. Rotational slide is the slide that landslide mass moves rotationally around the horizontal axis which occurs when a surface of rupture is curved concavely upward (Fig. 2.4A) while the translational slide is the slide that landslide mass moves along the surface with little rotation of backward tilting (Fig. 2.4B). The translational slide that the moving mass consists of a single or few closely related units moving down as a relatively coherent mass (Fig. 2.4C).

2.3.2 Falls

Falls are abrupt movements of masses of geologic materials, such as rocks and boulders that become detached from steep slopes or cliffs (fig. 2.4 D). Separation occurs along discontinuities such as fractures, joints, and bedding planes and movement occurs by free-fall, bouncing, and rolling (United States Geological Survey[USGS], 2013). Falls are strongly influenced by gravity, mechanical weathering, and the presence of interstitial water.

2.3.3 Topples

Toppling failures are distinguished by the forward rotation of a unit or units about some pivotal point, below or low in the unit, under the actions of gravity and forces exerted by adjacent units or by fluids in cracks(USGS, 2013) (fig. 2.4E).

2.3.4 Flows

There are five basic categories of flows that differ from one another in fundamental ways(USGS, 2013).

a. Debris flow

A debris flow is a form of rapid mass movement in which a combination of loose soil, rock, organic matter, air, and water mobilize as slurry that flows downslope (fig. 2.4F). Debris flows include < 50% fine grained materials. Debris flows are commonly caused by intense surface-water flow, due to heavy precipitation or rapid snowmelt that erodes and mobilizes loose soil or rock on steep slopes. Debris flows also commonly mobilize from other types of landslides that occur on steep slopes, are nearly saturated, and consist of a large proportion of silt- and sand-sized material(USGS, 2013). Debris-flow source areas are often associated with steep gullies, Debris-flow deposits are usually indicated by the presence of debris fans at the mouths of gullies.

b. Debris avalanche

This is a variety of very rapid to extremely rapid debris flow (fig. 2.4G).

c. Earthflow.

Earthflows have a characteristic "hourglass" shape (fig. 2.4H). The slope material liquefies and runs out, forming a bowl or depression at the head. The flow itself is elongate and usually occurs in fine-grained materials or clay-bearing rocks on moderate slopes and under saturated conditions(USGS, 2013).

d. Mudflow

A mudflow is an earthflow consisting of material that is wet enough to flow rapidly and that contains at least 50 percent sand-, silt-, and clay-sized particles. In some instances, for example in many newspaper reports, mudflows and debris flows are commonly referred to as "mudslides." (USGS, 2013).

e. Creep

Creep is the imperceptibly slow, steady, downward movement of slope-forming soil or rock. Movement is caused by shear stress sufficient to produce permanent deformation, but too small to produce shear failure. There are generally three types of creep: (1) seasonal, where movement is within the depth of soil affected by seasonal changes in soil moisture and soil temperature; (2) continuous, where shear stress continuously exceeds the strength of the material; and (3) progressive, where slopes are reaching the point of failure as other types of mass movements(USGS, 2013). Creep is indicated by curved tree trunks, bent fences or retaining walls, tilted poles or fences, and small soil ripples or ridges (fig. 2.4I).

2.3.5 Lateral Spreads

Lateral spreads are distinctive because they usually occur on very gentle slopes or flat terrain (fig. 2.4J). The dominant mode of movement is lateral extension accompanied by shear or tensile fractures. The failure is caused by liquefaction, the process whereby saturated, loose, cohesionless sediments (usually sands and silts) are transformed from a solid into a liquefied state. Failure is usually triggered by rapid ground motion, such as that experienced during an earthquake, but can also be artificially induced. When coherent material, either bedrock or soil, rests on materials that liquefy, the upper units may undergo fracturing and extension and may then subside, translate, rotate, disintegrate, or liquefy and flow(USGS, 2013). Lateral spreading in fine-grained materials on shallow slopes is usually progressive. The failure starts suddenly in a small area and spreads rapidly. Often the initial failure is a slump, but in some materials movement occurs for no apparent reason. Combination of two or more of the above types is known as a complex landslide.

2.4 Landslides and water

Slope saturation by water is a primary cause of landslides. This effect can occur in the form of intense rainfall, snowmelt, changes in ground-water levels, and water-level changes along coastlines, earth dams, and the banks of lakes, reservoirs, canals, and rivers(USGS, 2013). Land sliding and flooding are closely allied because both are related to precipitation, runoff, and the saturation of ground by water. In addition, debris flows and mudflows usually occur in small, steep stream channels and often are mistaken for floods; in fact, these two events often occur simultaneously in the same area. Landslides can cause flooding by forming landslide dams that block valleys and stream channels, allowing large amounts of water to back up. This causes backwater flooding and, if the dam fails, subsequent downstream flooding. Also, solid landslide debris can "bulk" or add volume and density to otherwise normal stream flow or cause channel blockages and diversions creating flood conditions or localized erosion. Landslides can also cause overtopping of reservoirs and/or reduced capacity of reservoirs to store water.

2.5 Landslides and seismic activity

Many mountainous areas that are vulnerable to landslides have also experienced at least moderate rates of earthquake occurrence in recorded times. The occurrence of earthquakes in steep landslide-prone areas greatly increases the likelihood that landslides will occur, due to ground shaking alone or shaking-caused dilation of soil materials, which allows rapid infiltration of water(USGS, 2013). The 1964 Great Alaska Earthquake caused widespread land sliding and other ground failure, which caused most of the monetary loss due to the earthquake. Other areas of the United States, such as California and the Puget Sound region in Washington, have experienced slides, lateral spreading, and other types of ground failure due to moderate to large earthquakes. Widespread rock falls also are caused by loosening of rocks as a result of ground shaking. Worldwide, landslides caused by earthquakes kill people and damage structures at higher rates than in the United States.

2.6 Factors controlling landslide

The failure of slopes in the terrain is steep and often occurs in conjunction with mostly flood. Cause of the failure slope includes natural factor such as caused by the topography, slope factor and the soil factor and another cause including human activities such as changes in soil slope to make agriculture and changes in land use. In 1961, Thailand had the forest area 183 million hectares, then decreased in 1974 to 119 million hectares and decreases again in 1996 to 80 million hectares. It can be clearly that, because the number of population increased, it has expanded of area to take advantage increase, making is access using areas that are steep slopes increased and have the opportunity to the affected because the failure of soil slope. The sediment resulting from the failure of soil slope is the major cause of a loss of life and property.

To recognize the reasons for the susceptibility of an area to sliding, and the factors which trigger the movement of the rock mass, is of extreme importance, because only a precise and correct diagnosis can serve as a basis for effective remedial measures. The variety of landslide types reflects the diversity of factors which are responsible for their origin. Factors producing slope movements are as follows:

1. The change of slope gradient.

The change of slope gradient may due to natural or artificial interference, i.e. to the undermining of the foot of the slope by stream erosion or by excavation. Exceptionally, the change of slope gradient may be produced by tectonic processes, by subsidence or uplift (Mencl and Záruba, 1969). The increase in slope gradient provokes a change of stress in the rock mass; the equilibrium is then disturbed by the increase in shear stress. Upon the relief of lateral stress the rocks on the slope loosen and facilitate the penetration of water.

2. Geologic condition (rock type and lineament zone)

Geologic conditions including types of rock and lineament zone are also the main factors influencing landslide formation. The granite and weathering residual soils from granite show the highest potential of landslide. Where on some sedimentary rock groups such as shale, mudstone, and siltstone influencing by geologic structures such as the bedding planes, joints, faults are second highest group (Mairaing, 2006).

3. The excess load by embankments, fill and waste dumps

The overloading may lead to an increase in shear stress and in the pore-water pressure of clayey rocks, which in turn produces a decrease in strength. The more rapid the loading, the more dangerous it is (Mencl and Záruba, 1969).

4. Effects of ground water

Ground water flow exerts pressure on soil particles, which impairs the stability of slopes and ground water can wash out soluble cement and thus weaken the inter granular bonds ; consequently, cohesion decrease and the coefficient of internal friction drops. The moving ground water washes out fine sand and slit particles from the slope and the underground cavities thus formed weaken the stability and confined ground water acts on the overlying impervious beds as uplift (Mencl and Záruba, 1969).

5. Shocks and vibrations

Earthquakes, large-scale explosions and vibrations of machines produce oscillations of different frequencies in rocks, and thus a temporary change of stress which can disturb the equilibrium state of the slope. In loess and loose sands, shocks can cause a disturbance of inter-granular bonds and consequently, a decrease in cohesion or internal friction. In saturated fine sands and sensitive clays, shocks may result in a displacement or rotation of grains leading to a sudden liquefaction of the soil (Mencl and Záruba, 1969).

6. Changes in water content

The effects of precipitation, Rain and melt water penetrate into the joints producing hydrostatic pressure; the increase in the pore-water pressure in soils induces a change of consistence, which in turn causes a decrease of cohesion and internal friction. Recurrent sliding movements generally occur in the years of unusually high rainfall and some authors who have measured the difference in electrical potential between two beds on the contact of which a sliding plane was formed explaining the increase in the water content inducing slope movements as due to electro osmotic processes. In period of drought clayey soils desiccate and shrink; as a result of this, fissure open, the cohesion of the soil diminishes and water can penetrate into the clayey rocks. Abrupt changes of water level (e.g. along the banks of reservoirs) may induce a displacement of grains, especially in fine or silty sand. A sudden increase in pore water pressure may result in a sudden liquefaction of soil (Mencl and Záruba, 1969).

7. Weathering of rock

Mechanical and chemical weathering gradually disturbs the cohesion of rocks. There are indications that in some landslides, chemical changes (hydration, ion-exchange in clays) induced by percolating water, are another deleterious factor. Thus, for instance, areas built up of clays and glauconitic sandstones show a susceptibility to sliding.

8. Changes in the vegetation cover of slopes

The roots of trees maintain the stability of slopes by mechanical effects and contribute to the drying of slopes by absorbing part of the ground water. The deforestation of slopes impairs the water regime in the surface layers.

The assumptions for the cause of increasing number of landslide for the past decade are 1) landslides naturally occur more often which might be related to the climate change, 2) mismanagement of land use due to the increasing number of population and the needs of land for producing agricultural products, that force people to stay in the landslide hazard areas.

2.7 Records of landslides in Thailand

The landslide that occurred in Thailand in the past tends to occur in places where the area is composed on hilly and forest. However, in the past twenty years, the over-exploitation in steeper sloping areas has increased. The events generated the loss of life in Thailand are shown in Table 2.1. For Thailand, the failure of soil slope has happen in natural and has long been occurred in hundreds or thousands of years ago. It left behind evidence of traces in many places of the alluvial fan as landslide scar.

Table 2.1. Records of landslide events in Thailand (30 years ago) (Mairaing, 2006).

Year	Locations	Live loss
November 1970	Tubsakae , Prachubkirikan	12
January 1975	Ronpibol, Nakhon Si thammarat	58
December 1982	Sibunpot, Phattalung	4
November 1988	Pipun, Nakhon Si thammarat	> 200
November 1988	Lansaka, Nakhon Si thammarat	12
August 1999	Kao-kichakud, Chantabuti	1
September 2000	Lumsak-Muang, Phetchabun	> 10
May 2001	Wangchin, Phae	> 30
August 2001	Lumsak, Phetchabun	132
May 2004	Mae Ramad, Tak	5
July 2004	Mae Aye, Chaingmai	1
May 2004	Mae Chame, Chaingmai	1
May 2004	Omkoi, Chaingmai	1
October 2004	Muang, Krabi	3
May 2006	Muang, Uttradit	71
May 2006	Bangtuk, Sukhothai	7
May 2006	Choehae, Phare	5
October 2006	Fang, Chaingmai	8

Thailand is located in the warm and tropical climate region. The tropical monsoons and typhoons from both the Andaman Sea and the South China Sea contribute to the heavy rain in the region. Rainy season starts in June commonly produces the over discharge in the basin locating in the northern part of the country. Not only the effect of heavy rain in the northern region, long and continue heavy rain in the southern Thailand ending season in December also cause the overbank full discharge in many places. The average annual rainfalls are ranging from 1000 to 1500 mm. for northern, northeastern and central parts of the country. At the eastern part and southern peninsular, the rainfalls are averaged from 2000 to 3000 mm. The rainfall induced landslides are normally occurred on the mountainous area due to single intent or long period rain. In case of prolong rainfall, the flood and debris flow will follow the landslides and cause more damage to the villages along flow passages and on the alluvial plain below. The most well-known landslide events in Thailand during the past decades are summarized as follows.

Southern Peninsular landslides

The landslide and flood events at Pipun and Lansaka districts, Nakhon Si Thammarat province due to heavy rainfalls in southern at November 1988 covered the area of more than 3,200 square kilometers and a number of casualties have reached 371 and caused the economic losses totally more than 7,000 Billion Baht. The impact of the flooding occurred between 18-21 November 1988 (Martin, 1989) due to heavy rainfalls in the Khao Luang mountain range. The damage was not caused by flooding or flooding overflow its banks only, but also included the huge movement of upstream sediments and forest to downstream area. Especially, debris flow at the Ban Kathun Nua, Pipun district, Khao Luang granite mountain with steep valley has shown the failure of natural slope become unstable in several locations.

The mega-scale tsunami-related earthquake on 26 December 2004 induced many subsequent small-scale landslides in southern Thailand (Tantiwanit, 2005 and Nawavitphaisit, 2005). Most people in many villages located within high elevation and high hill terrain of Phang Nga province were suffered from this natural hazard. In Phang Nga province, landslides have been recorded by (Akkrawintawong et al., 2008) using remote sensing and field information. They occurred where the mountain slope degree has from 30-40 degree. It was concluded that not only the heavy rainfall but also highly slopes and fractures in the granitic terrains may be much important physical factors.

Northern Thailand

The slope movement and flood events at Wang Chin District, Phrae Province occurred on 4 May 2001. This mega landslide was one of the country's worst record and landslide extended to Phee Pun Num mountain range covering Phrae, Lampang Sukhothai and a number of casualties has more than 30 peoples and caused the economic losses to more than 300 Million Bahts.

In May, 2004 in Mae Ramat District, Tak Province created the loss of 400 casualties, 2,500 damage houses, and 500 million Bahts. There were greater than 250 mm of precipitation prior to the landslide movement (Kosuwan, 2005). This is the main reason that people seem interested in the amount of rainfall as the main trigger mechanism for mass movement.

In August, 2001 in Nam Kor – Nam Chun area, Lumsak District, Phetchabun Province, create killed 150 persons and damaged 600 houses. The total amount of 645 million bahts was estimated for the economic lost. During that time, the amounts of rainfalls up to 150 mm were reported (Yumuang, 2007).

Eastern Thailand

The landslide and flood events at Chanthaburi province occurred where the area consists of the highly weathered granite rock basement with abundant joint and fractures. As the geological condition and setting is sensitive, heavy rainfall was also induced the slope failure to occur widely. In July 1999 and 2001, the landslide at Khao Khitchakood of Chanthaburi sub-basin brought a large amount of sediments, rock debris and fallen trees and transported them downstream to the coastal plain and river. This event has caused many casualties and the economic losses than 300 Million Bahts.

2.8 Researches of landslide relevant to this study

The study of engineering properties of soil for the stability analysis and study soil layer (overburden) overlaying on bed rock using by resistivity survey method has been carried out. Also, remote sensing techniques is often applied for landslide hazard zonation and remote sensing data that can be differentiated for the various phase within landslide study, such as, detection and classification of landslides.

The previous landslide area in Thailand and foreign countries was carried out. For example, Tantiwanit (1992) investigated the characteristics of landslides activities from the November 1988 storm event. " The study revealed that the significant factors controlling landslides could be summarized as follows: (1) residual soil from weathered granitic rocks was most susceptibility to landslide : (2) steep gradient over 30 percent; the change of vegetation cover to para-rubber plantations and (4) the triggering factor was highly rainfall intensity. "

Pantanahiran (1994) summarized the primary factors that controlled landslides in the Khao Luang Mountain Range during November 1988 storm as follows ; " (1) fractured limestone and granitic bedrock ; (2) shallow sandy soil from the weathering of granitic rock ; (3) steep slope of more than 30 degree ; (4) high rainfall in earlier November as well as particular storm in November ; (5) the pathway of storm ; (6) reduction in natural forest cover ; (7) planting of shallow root trees and crops and (8) recentness of clearing and replanting. Pantanahiran (1994) also used GIS and statistical technique to develop a landslide prediction model for Khao Luang Mountain Range. The model included eight parameters namely, elevation, aspect of slope, TM4 (Thematic Mapping Band- 4), flow accumulation, brightness, wetness, slope and flow direction. This model was capable of classifying 82 percent of landslides in the Tha Di stream basin at a 0.4 cutoff probability. "

Montovani et al (1996) summarized the feasibility and used for obtaining information needed for the approaches of hazard zonation using remote sensing techniques at the three different scales. " According to their work, landslide hazard mapping based on landslide inventory maps benefits most from information collected using remote sensing data, followed by heuristic approaches at regional and medium scales, statistical and landslide frequency analysis using indirect methods(for medium and large scale studies).” However, the spatial resolution of the most widely used satellite data (TM and SPOT images) are generally too coarse for landslide characterization unless the landslide is very large in size, or the image data is re -sampled and merged with other higher resolution satellite images (Rengers et al., 1992; Koopmans and Ferero, 1993; Singhroy, 1995). In present, the high spatial resolution satellite imagery from IKONOS, Quick bird, SPOT-5 is available for the production of landslide inventory maps. Some researches have been conducted using the 5.8 m resolution IRS-1D (Gupta and Saha, 2001), or simulated IKONOS data (Hervas et al., 2003).

The National Economic and Social Development Board (1997) conducted the study of natural hazard management in southern region of Thailand. “ The study consisted of 6 sub – topics as follows: (1) types of natural hazards and the affected area, (2) flood hazard and risk assessment, (3) soil erosion hazard and risk assessment, and (4) recommendation for natural hazard management in the southern region of Thailand. Geographic information system was also applied to manipulate, analyze and present in the study. ”

Thassanapak (2001) investigated the landslide assessment of Phuket Province using the influencing parameters of geology, landform, surface drainage zone, land -use and land cover, soil characteristics, and rainfall intensity. “ The relationship between this parameter and the spatial data were evaluated using the proposed weight-rating technique. The findings of this study revealed that most of the potential areas to be affected by very high and high susceptibility to landslide included the famous tourist resorts.”

Sasaki et al (2000) studied soil creep process and its role in debris slide generation field measurements on the north side of Tsukuba Mountain in Japan. “ Soil creep usually occurs during and after rain. The amount of creep caused by one rainfall has a positive correlation with the amount of rain and increase in soil moisture. For the valley head slope, soil creeps down and accumulates at the bottom bottle neck part of the head hollow, in which soil is compressed and forms a wavy landform. The lower end of the head hollow gradually becomes steeper and unstable. Side slopes along a valley-head slope show differences in topsoil thickness and creep speed between sections near and below a knick line. The amount of soil creep per year is larger in a section near a knick point than that below it. Soil gradually concentrates directly below the knick point, and the slope becomes steeper and unstable. The total amount of soil movement in 1 year was several millimeters for both the valley-head and side slopes. This means several liters/year of soil squeeze out for every slope section of 1 m in width. This value is similar to the general speed of erosion caused by debris slide (mainly topsoil failure, or shallow landslide) in Japan. Furthermore, neither surface erosion nor internal erosion has been observed in this site. Soil creep seems to carry most of the debris to places where debris slides will occur in this site. Soil creep prepares future debris slides and dominates the cycle of debris slide as well. ”

Lee and Min (2001) proposed the statistical analysis of landslide susceptibility at Yougin, Central Korea, in year 2000. “ Using a GIS and remote sensing, landslide locations were identified from interpretation of aerial photographs and field investigation. The relationship between landslide occurrence and cause factors were analyzed using probability, logistic regression, fuzzy logic, and neural network methods for landslide susceptibility assessment. Instability factors include surface and basement lithology and structure, bedding, altitude, seismicity, slope steepness and morphology, stream evolution, groundwater condition, climate, plantation area, land use, and human activities. The result of these studies is landslide susceptibility map ” .

Zomer et al. (2002) studied and used satellite remote sensing data for DEM extraction in complex mountainous terrain of the Makalu Burun National Park of Eastern Nepal. It is extremely useful for terrain analysis of topographic condition. Hydrologic model, automated stream and watershed delineation is easily facilitated by the extracted DEM. Three dimension terrains are able to visualization from DEM.

Thaijeamaree (2003) studied the stability of slopes in Nam Kor watershed area with special emphasis on soil engineering properties. Laboratory tests such as strength and other related engineering tests were conducted. The result from direct shear test shown that when the water content of the samples increasing, the shear strengths were decreased and the additional water gradually decreases the surface tension in the soil mass and in turn decreases the soil cohesion and friction angle. The fluctuation of moisture and groundwater regimes depends on the rainfall pattern and affect directly to the stability of soil slope. These relationships can establish the critical rainfall envelope when the Factor of Safety (F.S.) are equal to 1 with the various rainfall patterns from 1-14 raining days, the critical rainfall envelope can be established to use as future warning levels for the villager.

In the present Nan Province, there are many preliminary studies that had been done such as Environmental Geology Division, Department of Mineral Resources (2004) conducted a project of hazard zonation mapping from landslide in the whole Thailand in a scale 1:250,000 for identify the specific target areas to mitigate, monitor and improve. It was noted that the parameters that used in the model consisted of elevation, adjusted aspect, slope angle, water flow direction, and water flow accumulation, vegetation index from Landsat TM, soil characteristic (brightness), wetness and lithology. Landslide hazard zonation had been divided into 4 probability levels such as very high, high, medium and low. However, the proposed landslide hazard maps from the study should be investigated the accuracy and reliability in the field survey.

Mairaing and Thaijeamaree (2004) concluded that normally heavy rains on the mountainous area are among the triggering factors. " When rainy season started, unsaturated soil with high suction is still stable. The soil strength is decreased as the soil saturation increased due to percolating water. This behavior cause by the diminishing of the suction within the soil mass according to the theory of effective strength. Study of unsaturated soil strength in Thailand is very limited, thus the landslide on Nam Kor watershed area, Lom Sak District, Phetchabun Province is set for the study case. The undisturbed soil samples from various depths and geology were obtained. The soil strengths by multi-stage direct shear tests at the different water contents were performed. The results show that when percent of water contents in soil were more than

85, the soil cohesion is neglected. The soil strength can be represented by 3-D plane similar to Mohr-Coulomb Envelopes with the saturation ratio as the additional axis. This study also shows that the landslide prediction and warning can be done by using critical rainfall envelope calculated by various patterns of possible rainfall associated with stability analyses of the slopes on this location."

Singhroy and Molch (2004) concluded and mentioned the two different approaches that can be adopted for determining the characteristics of landslides from remotely sensed data. " The initial approach determines more qualitative characteristics such as number, distribution, type and character of debris flows. This can be achieved with either satellite or air born imagery collected in the visible and infrared region of the spectrum." The second approach complements are qualitative characterization, estimating dimension (e.g., length, width, thickness and local slope, motion and debris distribution) along and across the mass movement using stereo SAR, interferometric SAR and topographic profiles.

Kunsuwan (2005) studied the behavior for landslides on upstream areas in Chanthaburi river basin. As a result, landslide behaviors of Khlong Krating, Khlong Takhian and Klong Thung Phen in Chanthaburi sub-basin were compared in order to develop the warning criteria. The field investigation by soil sounding field tests and soil sampling were carried out on sites. The undisturbed soil samples were tested to obtain the soil shear strengths at the various control water contents as expected in the field during heavy rainfall. The Finite Element Method analyses were used to study the infiltration behaviors of rainfall to the soil. Then the hazard map is created by the relationship between the rainfall patterns, rainfall duration, return period, the slope stability and critical rainfall envelop in order to use for landslides warning. In general, the results show that the failure slopes are on the area of 25-35 degree slopes and the depth of 2.5 - 3.5 meters where the soil profiles are on the weathered granite rock with high natural water contents. The shear strength of soil decreased with increasing of the degree of saturation. At the shallow depth of soil, the changing of slope stability presented in teams of Factor of safety (F.S.) is consistent with the rainfall pattern.

Grammatical change of F.S. will occur at the soil depth between 0-3 meters, close the depth of the actual slide zone. The critical F.S. will occur at just right after the end of heavy rainfall. The correlation of the slope stability analyses with the historical rainfall data lead to landslide critical rainfall envelope of the F.S. equal to 1.1. From the geographic information system technique, the comparison between the generated hazard map and the historical landslide areas showed that 60.87% of past landslide areas are agreed with area of F.S. < 1.1. From the grain size distribution analysis, it can be suggested that the sediment of rock decreased with increasing of the distance from the source and can be able to predict in advance.

Local Government Office (2006) reported the general information of the existing risk areas from flooding and associated disasters in 9 Districts in Nan Province. " It noted that if there were continuing and heavy rainfall occurrence, the risk areas in Nan Province could be identified into two types: (1) the overbank flooded areas in the lower flood plain of Nan River and Nam Sa River, and (2) the flash flood areas in the areas of that lied on the canyon mouths of streams. The report also concluded that the major factors influencing flooding were as follows: a lot of sediments in the main rivers and canals, heavily rainfall, lack of enough water retentions and reservoirs, and the obstacle from the transportation routes. etc"

Yumuang (2006) studied the factors of the debris flow and debris flood (debris flow-flood) occurred in 11 August 2001 on the active Nam Kor alluvial fan in Phetchabun province, central Thailand. " Evidences of past activity registered in the alluvial fan, and the debris flow-flood event were reconstructed. The disastrous debris flow-flood event was not the work of the unusual high amount of rainfalls alone, as previously theorized, but is a work of combined factors from the terrain characteristics with specific land covers to the time delay for accumulation of debris and sediments. This combination of factors could lead to a debris flow-flood after a high amount of precipitation. The process could also be worse if a landslide formed a natural dam, then the dam was destroyed under the weight of impounded water. After such a disastrous event, it would take time for more plant debris and sediments in the sub-catchment area to accumulate before the next debris flow flood. ”

Soralump and Thowiwat (2007) studied about “ shear strength behavior of residual soils at various water contents that plays an important role on developing the warning criterion by water content or infiltrated water. Shear strength parameters are obtained by KU-MDS Shear Test which testing by varies degree of saturation, S_r in 3 levels: 60%, 80% and 100%. The test result showed a relationship in triaxis system of σ'_v, τ and S_r . The relationship will be used for modeling shear strength of soil and determining the API_{cp} for landslide warning. Furthermore, the residual soil strength parameters can be used for geotechnical engineering design. ”

Soralump et al (2007) concluded that “ soil shear strength is an important factor to indicate the landslide potential in various area of the country. However, due to the accuracy is depended on the size of target area, various shear strength testing method have to be design to match the accuracy and volume of test required. KU-MDS shear test and SRI test were used in this study. The result shows that KU-MDS is suitable for analyzing precipitation criteria for landslide warning. As for SRI test, it can be used as a landslide potential index since it indicates the shear strength reduction and void collapse behavior. ”

Akkrawintawong (2008) studied about landslide disasters at Nan province in northern Thailand because there was a series of landslides in August 23, 2006 that damaged properties and lives. The investigation was aimed to evaluate the susceptibility or hazard mapping of landslides in the Nan province using remote sensing interpretation and geographic information (GIS). Landslide locations or scars were identified in the Nan province from interpretation of aerial photographs as well as enhanced Landsat 7 ETM and IKONOS image for constructing the scar map. Then several factors affecting landslides, including elevation, slope aspect, land-use, lineaments and lithology were identified and display as maps using GIS. As a result, that the open-forest areas dominated by the steep slopes, sandstone, granite and tuffaceous rocks, proximal long faults/fracture, are prone to landslide hazard. Subsequently, degree of landslide hazards were expressed as very low to very high hazard levels. The landslide hazard map also indicates that high- to very high- level hazard areas is the mountainous areas in Chalerm Prakiat, Song Khwae, Chiang Klang, Thung Chang, and Pua Districts. Several districts, such as Na Noi, Wiang Sa and Nan in the southern part of the Nan province have much lower hazard level.

Giao et al. (2008) studied about weathering profile in a hilly granitic terrain at the Kata Noi beach in Phuket, Thailand. " A common geotechnical site investigation might not be suitable because it is difficult to bring a drilling rig to make the boreholes on the hillside. As an alternative electrical imaging (EI) survey was proposed. The main issues addressed in this study are how to find the boundary between the weathered and unweathered granites in a gradually changing resistivity pseudo-section. To solve these problems an approach was adopted, including (i) selection of a suitable EI array in the field; (ii) data analysis with topographical correction; and (iii) the incorporation of a conceptual weathering profile in EI data interpretation. As a result, a geological–geoelectrical model was constructed for the weathering profile of granitic rocks at the study site, which can be applied for investigation of similar hilly terrains in Southern Thailand. "

Lee et al. (2008) studied about resistivity image profiling surveys to develop the lithological and hydrogeological models of the subsurface in the southeastern part of Lishan landslide area of central Taiwan. " The bedrock consists of slate and rock samples were collected from boreholes. As a result, three electrical strata were recognized in colluvium. The shear zone is composed of shear gouges and shattered slate, and the undisturbed slate formation. The steep shear zone with resistivity ranging between 100~260 Ω-m, plays a crucial role in the local hydrogeological environment, because it forms a natural barrier which blocks and retains groundwater flowing down the slope."

Mairaing (2008) concluded that landslide problems and warning by geotechnical methods. A number of factors trigger the landslide including heavy rainfall during tropical monsoon, geological condition, and change of land uses. Prediction and warning by Geotechnical Engineering Method is the direct method for calculating the Factor of Safety (F.S.) of soil slope during heavy rainfall. Since, the strength of unsaturated soil is changed during the water infiltration in soil mass. Metric suction related to the volumetric water content in soil mass can be analyzed for each rainstorm patterns. Field monitoring can be confirmed by using tensiometer for pore pressure and moisture content measurement to verify the infiltration modeling. By applying the infinite slope stability analysis into geographical information system (GIS) approach, the prediction of landslide on Phuket area is given in term of the critical rainfall envelope. This method has the potential application for real time monitoring and warning in the future when the measuring rainfall intensity is re-ported from the automatic rain gage in the field. Then slopes stability condition can be calculated simultaneously and reported back for the further warning .

Soralump et al (2008) studied about " landslide hazard area in Doi Tung development project. Field and laboratory investigations were done including hand augers, test pits, drilling, landslide investigation permeability test and shear strength test. Stability analysis has been done using limit equilibrium concept. Landslide hazard area is assigned based on the slope degree which corresponds to slope factor of safety. As for land cover, considerations were made to include this factor in the analysis. Finally, verification was done and found good correlation between hazard area from the analysis and actual location of slope failure. "

Tapparnich and Jotisankasa (2009) concluded that " numerous shallow slope failures took place in residual soils derived from sedimentary rock formation of Uttaradit province in 2005 due to prolonged and intense rainfall. A representative slope in the landslide area has been instrumented for pore water pressure, rainfall intensity and slope movement for one rainy season in order to investigate the slope failure mechanism. Shear behavior of this material has also been investigated in details. Fully saturated consolidated-drained (CD) as well as suction-monitored direct shear tests have been performed on undisturbed samples collected from depths of 0.3-1 m. In addition, influence of number of drying /wetting cycles on saturated/unsaturated shear strength is investigated. The results from a simple infinite slope analysis suggested that the major slope destabilization mechanism is a combination of material degradation and pore water pressure increase. ”

Acharya et al (2009) studied the “ influence of shallow landslides on sediment using sandy soil. This work reported experimental measurements of sediment discharge after water-induced shallow landslides that are triggered on sandy soil in a flume under simulated rainfall. The principal aim of the research was to investigate how varying soil depth affects the location and occurrence of shallow slope failures, as well as how it affects sediment yields downslope. Four experiments were conducted using the same sandy soil and a 30° and 10° compound slope configuration under average rainfall intensity of 50 mm h^{-1} for up to 390 min. Soil depths were set to 200, 300, 400 and 500 mm. Engineering and geotechnical properties of the soil were examined. Sediment discharge and runoff were collected from the flume outlet at 15 minute intervals. Changes in the soil slope profiles after landslides and soil physical properties resulted from soil armoring, under continuous rainfall were also recorded. Results showed that sediment yields at the flume outlet, before landslides occurred, were very low and limited to the finer soil particles as would be expected for a sandy soil. However subsequent variations in sediment discharge were strongly related to failure events and their proximity to the outlet. Sediment yield was also affected by the original soil depth; the greater the depth, the higher the sediment yields. Post-failure reductions in sediment discharge were observed and attributed to post-failure slope stabilization under continuing rainfall and extensive soil armoring near the flume outlet. The results provide a clear linkage between landslides and sediment discharge due to hydrological processes occurring in the hill slope. This knowledge is being used to develop a model to predict sediment discharges from hill slopes following shallow landslide events.”

Soralump et al (2010) concluded that “ Landslide Risk Management of Patong City. Patong city located in Phuket province which is in the southern part of Thailand, it is one of most desire tourist destination in Thailand. The development of the city occasionally caused flooding and landslide. Geotechnical approach was used to produce landslide susceptibility map and landslide risk map. Buildings in the landslide hazard area were classified into various risk classes. Man-made factors such as the road, area of prohibition by law and others have been considered as a factor for landslide hazard mapping. Engineering practice handbooks were specification technique that will minimize the triggering of landslide. Critical API was calculation and the appropriate warning method was adapted for local personnel to use as critical for landslide warning. All the works mentioned were closely done together with the Patong Municipality in order to customize these mitigations for the person who are in charge. The project was one of pioneer works on the first systematic landslide risk mitigation in Thailand. ”

Chapter III

METHODOLOGY

3.1 Introduction

The chapter III presents the methodology all about data collection associated with landslides, including technical aspect of remote sensing and geographic information systems, 2-D electrical resistivity survey, method of multi-electrode resistivity imaging in the study area, depth of penetration in electrical imaging profiles and the operating procedure in resistivity imaging techniques. In addition, soil sampling method including the collected and disturbed and undisturbed soil samples in the study area. Furthermore, testing disturbed and undisturbed soil samples to find the physical properties and engineering properties in geotechnical engineering laboratory, classification of soil by unified classification system and slope stability analysis and Factor of safety were carried out.

Generally, the factors related to landslides can be subdivided into 3 groups based on Van Western et al. (2008), namely triggering mechanism, human activities, environment factors. Assessing relative landslide hazard is the objective of the method described in this chapter. The flow chart of thesis methodology is shown in figure 3.1.

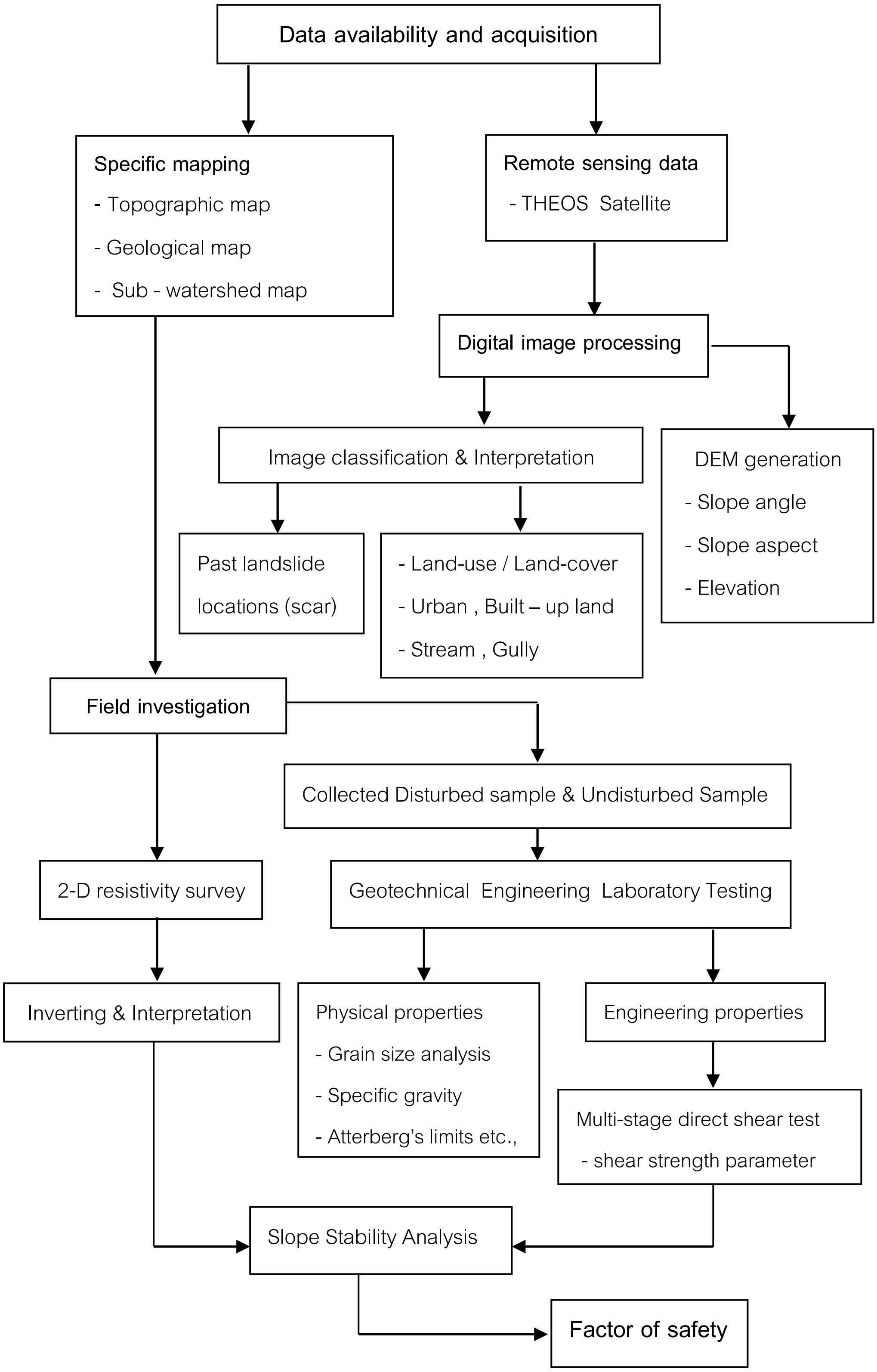

Figure 3.1 Flow chart of thesis methodology used in this research

3.2 Technical aspects of remote sensing

Remote sensing refers to specific uses for obtaining information about the Earth' s surface, which sense electromagnetic (EM) radiation (Gupta, 2012). Therefore, data and image are obtained only from the earth surface. Remote sensing is an excellent tool for site characterization because it is not limited by extremes in terrains or hazard conditions, which may be encountered during an on-site appraisal. These are effective for basic and apply research covering a wide range of subject, including mineral exploration, geo-environmental and geo-hazard.

Remote sensing data should be acquired into early stages of the investigation and used in conjunction with traditional mapping techniques (Akkrawintawong, 2008). It is the best suited method for the following purposes.

1. Preliminary assessment and site characterization of an area prior to the application of move costly and time-consuming traditional assessment techniques, such as field survey mapping and geographic survey.
2. Clarification of geo-scientific problem using the broad perspective provided by an aircraft or satellite image.
3. Geo-scientific assessment of regions with limited or no access, such as rugged terrain, hazard sites and disaster areas.

The high resolution satellite image data, such as IKONOS, QUICK BIRD, WORLD VIEW, THEOS can be used for the scale of 1:50,000 to 1:10,000 (Temesgen et al., 2001). These data are commonly used to characterize natural resources that have a wide distribution (e.g. tropical rain forests), to monitor flooding as well as detect and monitor environment problems (e.g. impacts on soil and ground water, land subsidence, hazard due to landslide, forest fires and soil spills). Satellite images have also been shown to be an effective tool for characterizing and assessing areas of human activity including deforestation, cut natural slope, extension of land development areas. The factors that determine the utility of remote sensing in landslide hazard assessments are scale, resolution, and tonal or color contrast of the data. Resolution of satellite image is determined by size and numbers of picture elements or pixel used to form an image. The smaller pixel size of an image is the greater the resolution of the data. For example, the pixel size of an image 30 m x 30 m is low resolution than 5 m x 5 m pixel size of an image (Akkrawintawong , 2008). Spectral resolution also needs to be taken into consideration when selecting the type of data since different sensors are designed to cover different spectral regions.

This research outline the scope of a typical remote sensing work , starting with definitions and goals of remote sensing, covering digital data rectification and enhancement techniques, and describing data interpretation and map production approaches, referring to the Amphoe Chiang Klang, Changwat Nan.

3.2.1 Remote sensing information

Remote sensing information is one of the most important data for landslide assessment. It is very useful in detecting and mapping landslide scars, land-use/land cover, agriculture land, forest land, topographic and geological conditions (Akkrawintawong, 2008). The satellite images are used widely as fundamental knowledge for land planning, land use mapping, geological field survey, and landslide scar detection. The author applied data of the THEOS from Geo-Informatics and Space Technology Development Agency (GISTDA) for analyzing and interpreting landslide scar and land-use/land cover, agriculture land, forest land, urban and building land location. The study area under investigation lies within the latitude 19º 26′ 39″ N , 19º 24′ 38″ N , 19º 13′ 29″ N, 19º 11′ 35″ N and longitudes 100º 58′20″ E , 101º 11′ 34″ E, 100º 55′ 10 ″ E, 101º 08′ 24″ E

Table 3.1. Technical parameters of the THEOS satellite remote sensing systems (GISTDA, 2012).

List	THEOS
Nominal altitude	822 km
Weight	715 kg
Size	2.1 m x 2.1 m x 2.4 m
Estimates life time	5 year
Memory size	51 GB
Argument of perigee	90 degree
Ground velocity	6.6 km/sec
Revolution period	101 min
Number of revolutions per day	14+ 5/26
Orbit full cycle	26 day
Ground resolution	2 m (PAN)
Field of view	22 km
Number of point (pixel) / picture	12,000 point / picture
Accessible corridor	1,003 km (± 30º) 2,273 km (± 50º)

THEOS-1 imagery : The THEOS-1 commercial remote sensing satellite images consist of panchromatic (white and black) image data of Earth' s surface. The powerful remote sensing system has changed the way the public and private industry worldwide view the Earth's surface and plan for its future. The images offer stunning new information to farmers, city planner, environmental planner, realtors, geologists, the media and others. THEOS satellite image with the resolution of 2 meter provided by Geo-Informatics and Space Technology Development Agency (GISTDA) for locating landslide scar and land-use/land cover, agriculture land, forest land, urban and building land locations precisely.

Table 3.2. Spectral bands in THEOS satellite imagery (GISTDA, 2012).

Band	Multispectral	Panchromatic
1 (Blue)	0.45- 0.90 μm	0.45-0.90 μm
2 (Green)	0.53- 0.60 μm	-
3 (Red)	0.62- 0.69 μm	-
4 (NIR)	0.77- 0.90 μm	-

Remarks : spatial resolution : 2 m for panchromatic(PAN) , 15 m for multispectral

3.2.2 Non- remote sensing data

Non-remote sensing data comprise all maps from available data sources and field investigation, which are related to the study area. Import maps for this research are topographic maps, land-use maps, geological maps, Stream order maps, Sub-basin maps, slope maps (Table 3.3). Additionally, other relevant reports and documents collected from concerning organizations are also essential for the analysis.

Table 3.3. Overview of non-remote sensing data types and sources for this study.

Data types	Scale	Original of data format	Sources	Number of maps
Geologic map	1:50,000	Hard copy	Department of Mineral resources	2
Land use map	1:25,000	Shape file of ArcGIS	Land Development Department	1
Topographic map	1:50,000	Hard copy	Royal Thai Survey Department	2
Stream order map	1:50,000	Shape file of ArcGIS	Department of Mineral resources	3
Slope aspect map	1:50,000	Shape file of ArcGIS	Department of Mineral resources	2

3.3 Technical aspects of Geographic Information system

Geographic information systems (GIS) are computer-based systems for data capture, input, manipulation, transformation, visualization, combination, query, analysis, modeling and output, with its excellent spatial data processing capacity. In addition, it has been used widely in landslide hazard assessment (Carrara et al., 1999). GIS is very useful tool for spatially distributed data processing and analysis. Conceptually, GIS should be able to utilize spatial data in any form, whether raster, vector or tabular. GIS provides the following tasks of capabilities to handle geo-referencing data.

Data Input : Data input components convert data from their existing from into the other one that can be used by GIS. The input data are usually derived from available data assessment during field visits, and significantly supported by satellite image interpretation. Geographic reference and attribute data must be entered into GIS. Geographic reference data are coordinates (either in terms of latitudes and longitudes or columns and rows), which give the locations of information being entered. Attribute data are associated with a numerical code to each cell or set of coordinates and for each variable, or to geo-information (land-use, vegetation type, and rock type, etc.)

Data Structures : The input data from the earlier step are needed to store in GIS as a spatial database. The spatial data (vectors and raster model) are structured and organized within the GIS according to their location, interrelationship, and attribute design as a systematic database of analysis. **Vector model**: The vector model represents all information as points, lines and polygon, assigns as a unique set of x, y coordinates to each piece of information (figure 3.2 , 3.3) Vector data can offer a large number of possible overlay inputs or layers of data. The vector model does represent the map area more clearly than a raster model. **Raster model**: The raster model use grid cells to reference and store information. The spatial data map is divided into a grid or matrix of square cells identical in size, and information attribute of the database (figure 3.4). A cell can display either the dominant feature found in that cell or percentage distribution of all attributes found in the same cell.

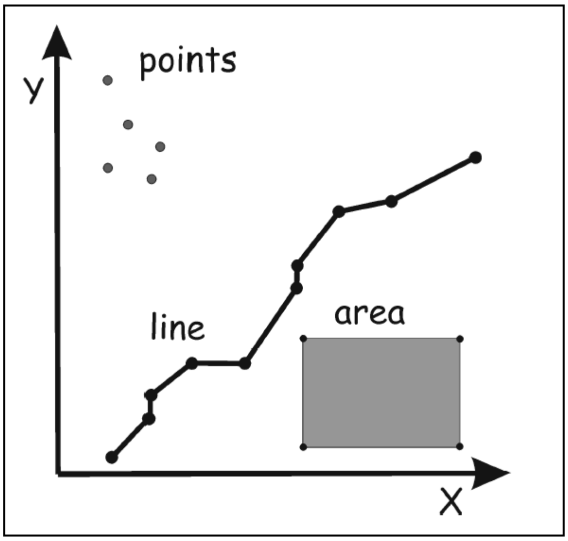

Figure 3.2. Vector data of points and line with x,y coordinate

(http://www.indiana.edu/~gisci/courses/g338/lectures.html).

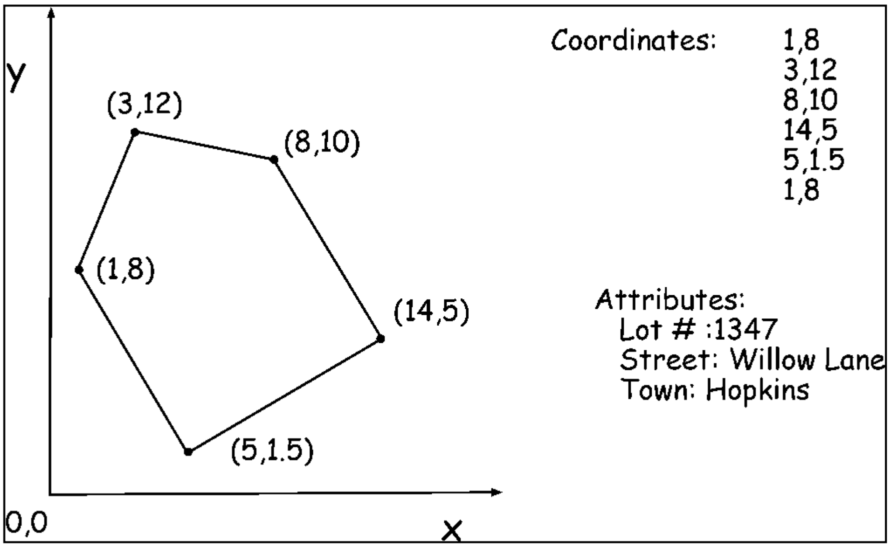

Figure 3.3. Vector data of line and polygon with x,y coordinate

(http://www.indiana.edu/~gisci/courses/g338/lectures.html).

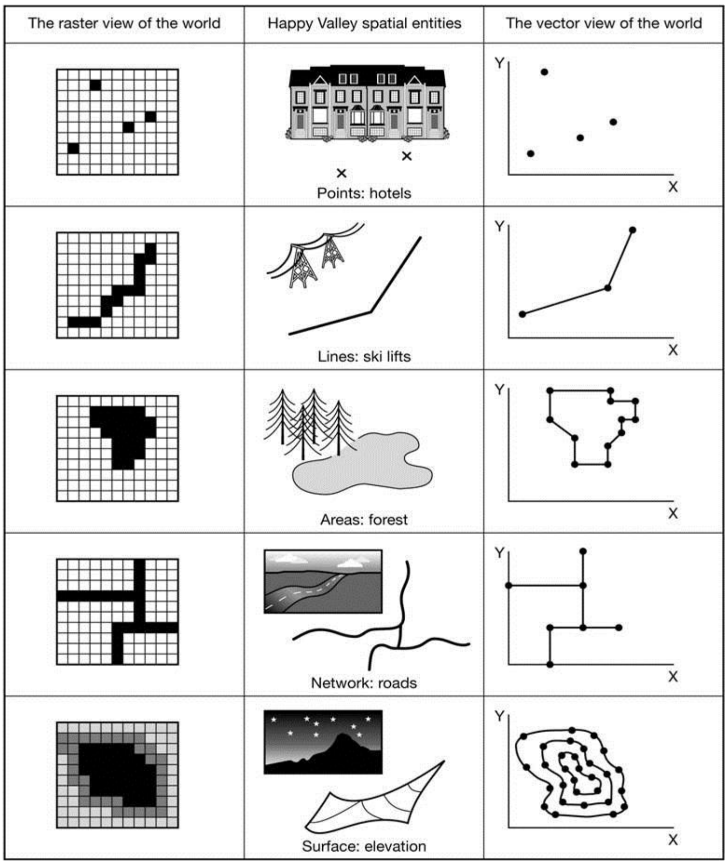

Figure 3.4. A graphical comparison of vector data and raster data structure (http://www.indiana.edu/~gisci/courses/g338/lectures.html).

The establishment of a database of information relating to landslides is a major task involving various inputs. The input data are usually derived from available data, assessment during field visits, possible supported by satellite image interpretation. Two different types of data must be entered into the GIS, geographic references and attributes. Geographic reference data are the coordinates (either in terms of latitude and longitude or columns and rows) which give the location of the information being entered. Attribute data associate a numerical code to each cell or set of coordinates and for each variable, or to represent actual values (e.g., 900 m elevation, 25 degree slope gradient) or to connote categorical data types (land-uses , vegetation type, land cover, rock type, etc.). Data manipulation and processing are performed to obtain useful information from a systematic database in the GIS system (Akkrawintawong, 2008). These are the operations using analytical techniques to answer specific question formulated by the user. The manipulation process can range from the simple overlay of two or more maps to a complex extraction of output data formats that include maps, graphs, reports, tables, and charts, either as a hard-copy , as an image on the screen , or as a text file that can be carried into other software programs for further analysis.

3.3.1 Integrating remote sensing and GIS for geo-spatial analysis

Data integration techniques for landslide hazard assessment are based on two important assumptions (Ward et al ., 1993) as follow : 1) the occurrence of the past landslides in the area are dependent on the input spatial geo-scientific data, and 2) the future landslides will occur under similar conditions in which the past landslides have occurred. The concept of integrate landslide assessment analysis requires designing a database system as a basis for landslide hazard analysis. The parameters relevant to the landslide hazard phenomena were grouped into thematic categories. Subsequently, there is need to quantitatively describe the landslides and their attributes, identifying the data categories and general data groups with variables that could be mapped from remote sensing imagery. There are various integration methods which can be utilized to combine spatial data from diverse sources together, to describe and analyze interactions, to make predictions and to prepare landslide hazard zonation map.

3.4 Resistivity survey method

Resistivity measurements may be made at the Earth's surface, between boreholes or between a single borehole and the surface. With special cables measurements can be made underwater in lakes, rivers and coastal areas. The following basic modes of operation can be used ; profiling(mapping), vertical electrical sounding(VES), combined sounding and profiling(two-dimensional resistivity imaging), and electrical resistivity tomography(ERT) (Lange and Seidel, 2007).

Resistivity survey methods can be applications to find : Investigation of lithological underground structures, estimation of depth, thickness and properties of aquifers and aquicludes, determination of the thickness of the weathered zone covering unweathered rock, detection of fractures and faults in crystalline rock, Resistivity methods are useful only if the resistivity of the target differs significantly from the resistivity of the host material. For the successful planning of any geo-electrical survey and for the interpretation of the data, it is very important to know the resistivities of the materials in the study area. Resistivities of some selected materials, including typical domestic and industrial waste, are listed in Table 3.4

Table 3.4. Resistivity values for geological and waste materials (Lange and Seidel, 2007)

Material	Resistivity (in Ωm)	
	minimum	Maximum
Gravel	50 (water saturated)	$>10^4$ (dry)
Sand	50 (water saturated)	$>10^4$ (dry)
Silt	20	50
Loam	30	>1000
clay (wet)	5	30
clay (dry)		25
peat , humus , sludge	15	25
Sandstone	< 50 (wet , jointed)	$>10^5$ (compact)
Limestone	100 (wet , jointed)	$>10^5$ (compact)
Schist	50 (wet , jointed)	$>10^5$ (compact)
Igneous and metamorphic rock	< 100 (weathered , wet)	$>10^6$ (compact)
rock salt	30 (wet)	$>10^6$ (compact)
domestic and industrial waste	< 1	> 1000 (plastic)
natural water	10	300
sea water (35 % NaCl)	0.25	
saline water (brine)	< 0.15	

Pore fluids considerably reduce the resistivity of porous sediments (Table 3.4). The resistivity of a rock that is saturated with highly mineralized water can be significantly lower than given in Table 3.4. Sandy sediments containing highly mineralized pore water can have the same resistivity as clay. In such cases it is difficult to distinguish between sand and clay layers on the basis of resistivity alone. Data from a combination of methods (e.g., induced polarization, seismics, ground penetrating radar) should be used to identify subsurface lithology and structures (Lange and Seidel, 2007).

3.5 2-D resistivity survey

In the geoelectrical investigation of disaster problems, 2-D resistivity surveys (2-D imaging) have played an increasingly important role in the last few years. The advantages of 2-D measurements are their high vertical and lateral resolution along the profile, Comparatively low cost due to computer driven data acquisition, which means only a small field crew is needed (one operator and, for basic electrode spacing $\geq$ 5 m, one assistant) (Lange and Seidel, 2007). Owing to the performance capabilities of the available instruments , 2-D surveys are, In general, limited to investigation depths down to about 100 m. 2-D measurements are often the most suitable geophysical method for solving disaster problems and environment problems, which are frequently related to 2-D or 3-D resistivity features. The aim of a geoelectrical survey at the Earth surface is to provide detailed information about the lateral and vertical resistivity distribution in the ground. As the principles applied are similar to those of computer tomography in medicine, such geoelectrical investigation are also called impedance tomography or 2-D resistivity imaging(Lange and Seidel, 2007). The result of a 2-D resistivity survey is a cross-section of the calculated rock resistivities along the profile line. This cross-section should include the structural interpretation of the resistivity data (e.g., the groundwater table, the thickness of the weathered zone covering unweathered rock, detection of fractures and faults in crystalline rock) (Lange and Seidel, 2007).If the spacing between profiles is not too large, horizontal resistivity sections for different depths can also be derived from the data.

3.5.1 Multi-electrode resistivity imaging for environmental application

For a few years, the evolution of electronic components and of computer processing have permitted to develop field resistivity equipment (SYSCAL Switch and SYSCAL Pro Switch units) which includes a large number of electrodes located along a line at the same time, and which carries out an automatic switching of these electrodes for acquiring profiling data (Bernard et al., 2006). The apparent resistivity pseudo sections measured with such a technique are processed by an inversion software which gives interpreted resistivity and depth values for the anomalies detected along the profile. The multi-electrode resistivity technique consists in using a multi-core cable with as many conductors (24, 48, 72, 96, …) as electrodes plugged into the ground at a fixed spacing, every 5 m for instance. In the resistivity meter itself are located the relays which ensure the switching of those electrodes according to a sequence of readings predefined and stored in the internal memory of the equipment.

The 2-D resistivity images obtained with such a multi-electrode technique are used for studying the shallow structures of the underground located a few tens of meters down to about one hundred meters depth; these images supply an information which complements the one obtained with the more traditional Vertical Electrical Sounding (VES) technique, which mainly aims at determining the depths of horizontal 1D structures from the surface down to several hundred meters depths. Several examples are presented for various types of applications(Bernard et al., 2006): groundwater (intrusion of salt water in fresh water), geotechnical (detection of a fault in a granitic area), environment (delineation of a waste disposal area), archaeology (discovery of an ancient tomb) and mineral exploration (detection of a metallic orebody).

3.5.2 Method of multi-electrode resistivity imaging in study area

In August 23, 2006, a landslide was initiated in many areas of Nan province after five days of the unusual heavy rainfalls. This event caused many houses, buildings and bridges completely destroyed of several districts including Charoem Phrakiat, Thung Chang, Chiang Klang, Bokuai (Akkrawintawong, 2008). Therefore, Ban Nong Pla village, Amphoe Chiang Klang, Changwat Nan, Thailand was selected for landslide hazard investigation. Ban Nong Pla village located on the valley with steep slope and high elevation with high of 700 -1200 meters above mean sea level. The development tends to move higher to the mountain and changing of the land use and land cover caused the landslide in the area. Therefore, this research will use 2-D resistivity survey to find study the soil layer (overburden) overlaying on bed rock (estimation the depth of soil layer) and determination of the thickness of the weathered zone covering unweathered rock shown in figure 3.6 and position of survey line have three line included BNP-1 , BNP - 2 , BNP-3 shown in figure 3.7 with the details such as BNP-1 make the line from the southeastern to northwestern(SE – NW), BNP – 2 make the line eastern to western (E – W) and BNP – 3 make the line southeastern to northwestern(SE – NW), respectively. The thickness of soil overlying on rock basement is important for calculating the volumetric of possible collapse.

This technique, called Resistivity Imaging or Electrical Resistivity Tomography (ERT), finds applications in the environment, groundwater, civil engineering and archaeology fields. The images which are obtained (apparent resistivity pseudo sections) are processed by an inversion software which gives interpreted resistivity and depth values for the anomalies detected along the profile(Bernard et al., 2006). The multi-electrode resistivity technique consists in using a multi-core cable with as many conductors (24, 48, 72, 96, …) as electrodes plugged into the ground at a fixed spacing, every 5 m for instance (figure 3.5). In the resistivity meter itself are located the relays which ensure the switching of those electrodes according to a sequence of readings predefined and stored in the internal memory of the equipment. The various combinations of transmitting (A,B) and receiving (M,N) pairs of electrodes construct the mixed sounding / profiling section, with a maximum investigation depth which mainly depends on the total length of the cable.

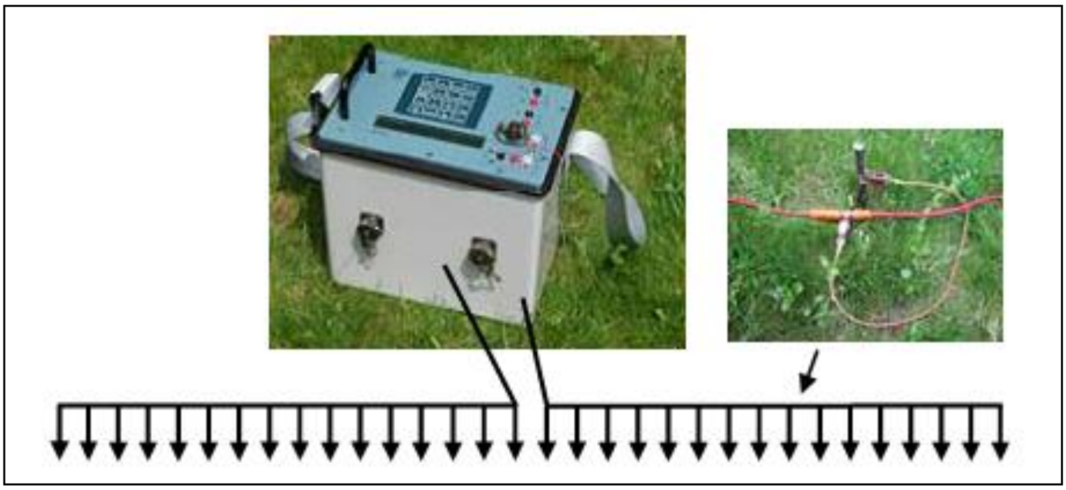

Figure 3.5. SYSCAL switch multi- electrode equipment

Figure 3.6. 2-D resistivity survey in study area using 48 electrodes (SYSCAL switch multi- electrode equipment)

Various types of electrode combinations can be used, such as Dipole-Dipole, Wenner-Schlumberger, Pole - Pole arrays. Each type of combination has advantages and limitations in terms of lateral resolution and vertical penetration for instance, as summarized in figure 3.8.

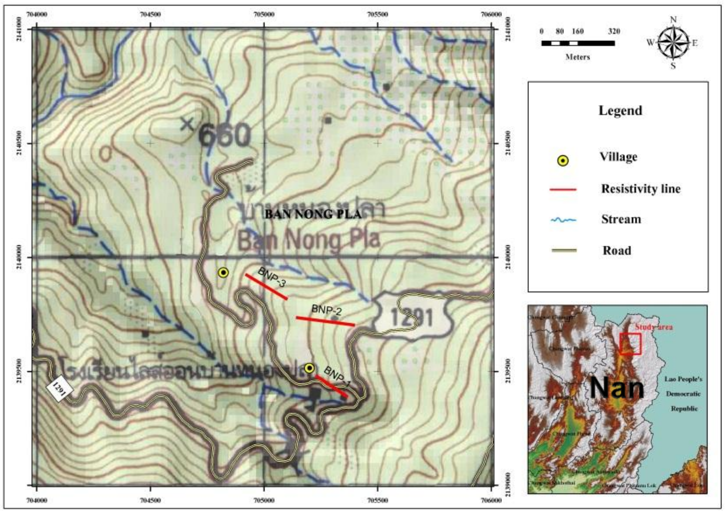

Figure 3.7. Position of 2-D resistivity line survey in study area.

Arrays		DIPOLE - DIPOLE	WENNER - SCHLUMB	POLE - POLE
Main criteria	Resolution Depth Field set-up	best weak regular	regular regular regular	weak best weak
Other criteria	Amplitude Natural noise Coupling noise	weak regular best	regular regular regular	best weak weak
CONFIGURATION ESTIMATED INVESTIGATION DEPTH		A B M N a na a L about 0.2 x L	A M N B na a na L about 0.2 x L	B A M N a L about 0.9 x L

Figure 3.8. Properties of electrode arrays 2-D.

For instance, for a 48 electrode 5 m spacing Pole Dipole array, a required quality of 1% in a standard noise, a 100 ohm.m average resistivity and a 2.5 kohm ground resistance, the duration of acquisition of 500 readings is 25 minutes with a 600 V output voltage one channel unit (SYSCAL R1 Plus Switch 48 shown in figure 3.9.), while it decreases down to only 2 minutes with a 800V output voltage ten channel unit (SYSCAL Pro Switch 48).

Figure 3.9. SYSCAL R1 Plus Switch 24 - 48 or 72 electrodes spacing between electrodes (std) : 5m (using to 2-D resistivity survey in study area)

3.5.3 The operating procedure in resistivity imaging techniques

The procedure for getting resistivity imaging includes the following four successive steps (4 steps) (Bernard et al., 2006) :

1. Creating the sequence of measurements with *ELECTRE II SOFTWARE* ; the sequence depends on the number of electrodes, their spacing, the type of array (Schlumberger - Wenner, Dipole-Dipole, Pole Dipole, Pole Pole…), the investigation depth to reach; loading of this sequence into the memory of the resistivity meter.
2. Taking the readings in the field with *SYSCAL SWITCH EQUIPMENT*, after the electrode resistance checking, and the introduction of the stack number which depends on the signal to noise ratio. During the measurements, the output voltage of the equipment is automatically adjusted to the level of the signal measured.
3. Transferring and process the data with *PROSYS SOFTWARE* from the memory of the equipment to a PC, filtering of noisy data in relation with their standard deviation or on

the level of the signal, introduction of the topography (electrode elevation), visualization of the results by level of investigation depth.

4. Inverting and interprete the data with *RES2Dinv SOFTWARE,* which, after a certain number of iterations, gives the values of the interpreted resistivities (through a color scale), and depths.

Finally, The 2-D resistivity images obtained with such multi-electrode techniques are used for studying the shallow structures of the underground located a few tens meters down to about one hundred meters depth; these images supply an information which complements the one obtained with the more traditional Vertical Electrical Sounding (VES) technique(Bernard et al., 2006) . The automatic processes in the acquisition and the inversion of the data which simplify the work of the operators, but do not replace them for controlling the quality of the measurements and for managing the equivalence properties during the interpretation phase.

3.5.4 Examples of application of resistivity for environmental studies

The multi-electrode resistivity imaging technique can be used in many applied geophysics domains such as Hydrogeology, Geotechnical, Archaeology, Environment, Mining exploration(Bernard et al., 2006).

1. Hydrogeology: intrusion of salt water within sediments

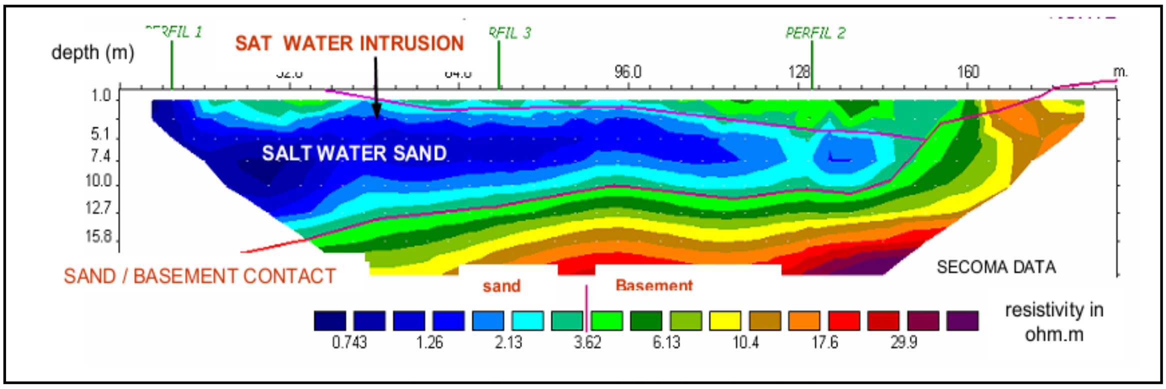

Figure 3.10. Salt water resistivity imaging in Spain (Loke, 2000)

In the management of the coastal aquifers, the intrusion of salt water coming from the see represents one of the major issues for the long term evolution of the fresh water resources. Through the important contrast of resistivity between salt water and fresh water, resistivity imaging permits to follow the limit between both types of waters at places where no drillhole is available(Bernard et al., 2006). In the example of figure 3.10. obtained in Spain (Wenner-Schlumberger array), salt water sands feature less than 1 ohm.m of interpreted resistivity, while the non invaded zone gives more than 20 ohm.m.

2. Geotechnical engineering : localization of a fault in a granitic area

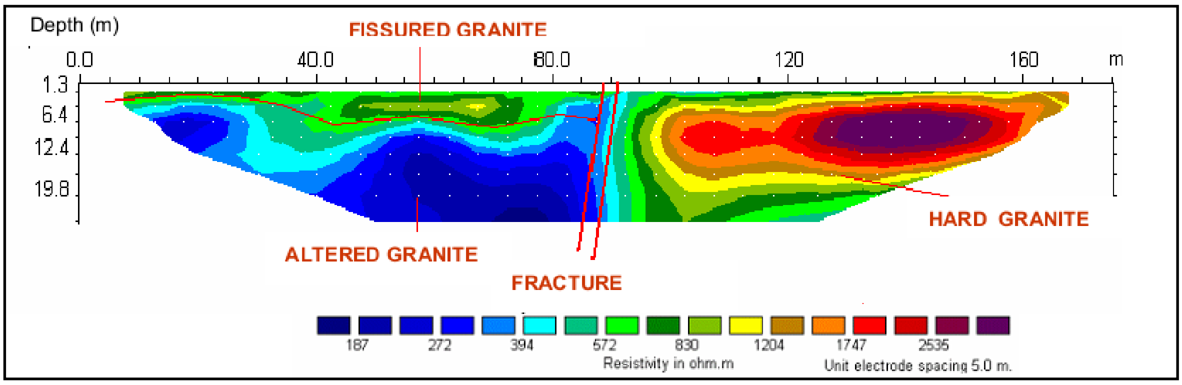

Figure 3.11. Fault resistivity imaging in Spain (Loke, 2000)

In linear civil engineering works (roads, railways, …), the detection of geological structures gives an important piece of information before control drillholes(Bernard et al., 2006). In the example presented in figure 3.11. A hard granite area is characterized by a value of interpreted resistivity greater than 2000 ohm.m, while alteration is under 200 ohm.m. A fault, clearly seen on the image separates both types of structures.

3. Archaeology : detection of cavities close to the surface

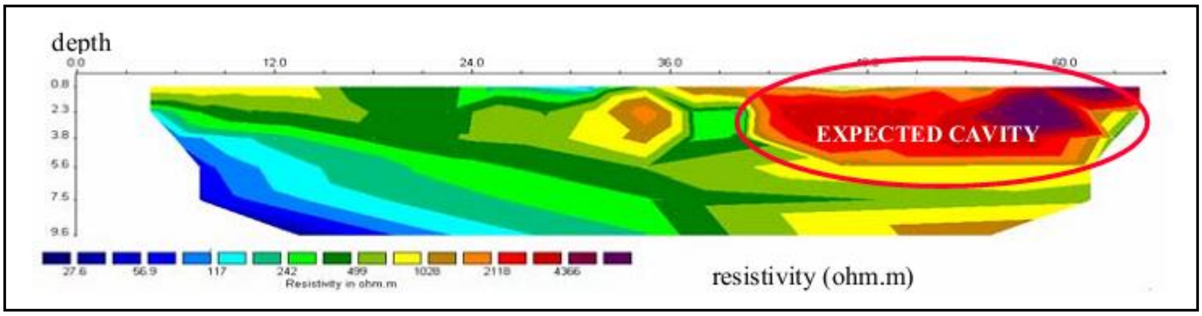

Figure 3.12. Cavity resistivity imaging in Middle East (Loke, 2000)

In civil engineering as well as in archaeology, the detection of cavities represents an important activity. A cavity filled with air located above the water level appears as a resistive object with respect to the surrounding formations(Bernard et al., 2006). In the example of figure 3.12. A set of 24 electrodes spaced at 3 m has been used to detect an ancient tomb located within the first four meters depth: its interpreted resistivity is greater than 2000 ohm.m, while the sand and gravel layers remain lower than 500 ohm.m.

3.6 Method of collected soil sample in study area

Test pits, trenches and shallow excavations are often used in site investigations, particularly during Investigations for low- and medium-rise construction, because they provide an economical means of acquiring a very detailed record of the complex soil conditions which often exist near to the ground surface. It is worth remembering, however, that trial pits and other exposures can also be used for in situ testing and to obtain high-quality samples. In order to consider geotechnical engineering data for the landslide hazard analysis, three properties of residual soil were investigated: 1. strength reduction due to increasing of moisture content 2. soil plasticity and 3. grain size distribution. Samples of residual soil were collected by KU - miniature sampler (shown in figure 3.13 (a), (b). The soil sampler was designed to ease the sampling of the residual soil. especially in the landslide area (Mairaing et al., 2005). Furthermore, area ratio of the sampler is about 18 percent, this study to ensure the limited disturbance of soil samples.

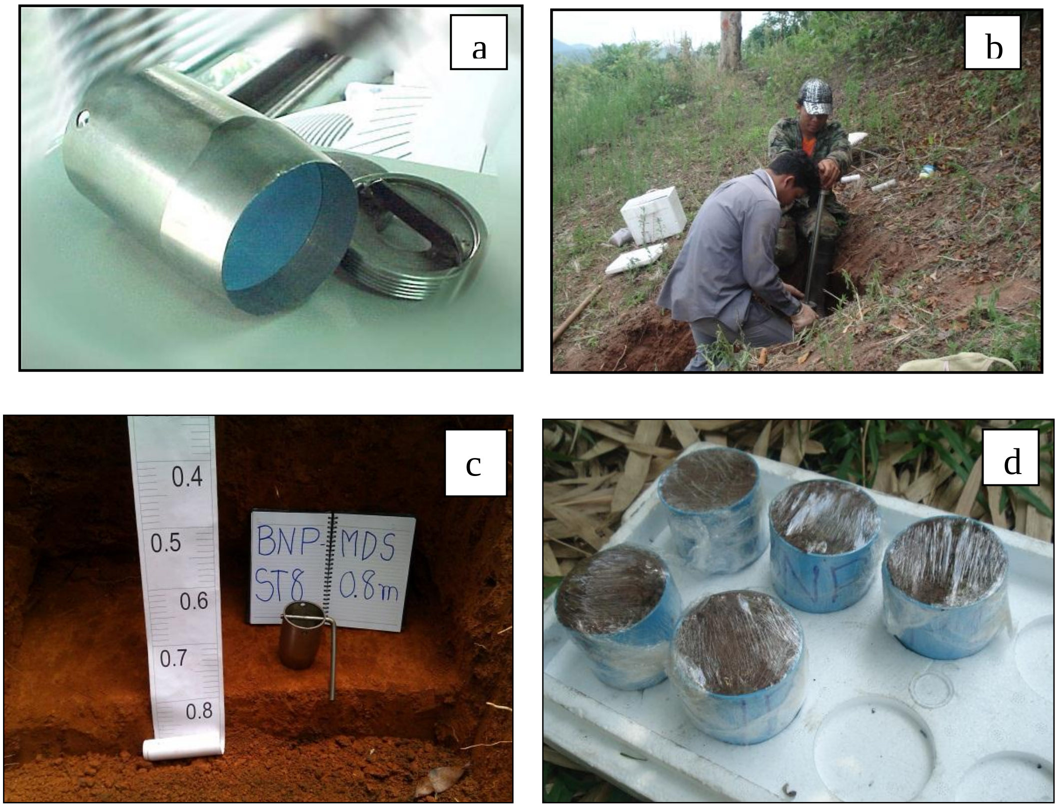

Figure 3.13. KU - miniature sampler (a),(b) and The collected method the Undisturbed soil sample(c), (d) (Mairaing et al., 2005).

3.6.1 The weathering profile of soil and rock

While rocks are exposed to weathering processes, a series of weathering stages before converted to residual soils. The weathering stages typically manifested within a soil/rock profile from the ground surface to unweathered rock are follows (Geological Society Engineering Group, 1990)

1. Residual soil (Stage VI) that is the original structure of the material has been destroyed by pedological process. The rock is completely changed to a soil in which the original rock texture has been completely destroyed (see figure 3.14).

2. Completely weathered (Stage V) that is the rock material has been changed to soil in which the original rock texture is (mainly) preserved.

3. Highly weathered (Stage IV) that is more than half of the rock material has decomposed or disintegrated into soil. Fresh rock is present as block or rounded core-stones (50 – 100 % soil from decomposition of the rock mass).

4. Moderately weathered (Stage III) that is less than half of the rock material has decomposed or disintegrated into soil. Fresh rock is present as blocks or core-stones that fit together (Up to 50 percent soil from decomposition of the rock mass).

5. Slightly weathered (Stage II) that is discoloration on the rock surface indicates weathering, especially along discontinuity surfaces and weathering has not penetrated fractures and joints(100 % rock ; discontinuity surfaces or rock material may be discolored).

6. Unweathered (Stage I) that is 100 % rock (Fresh rock), no visible sign of rock material weathering except perhaps slight discoloration along discontinuity surfaces.

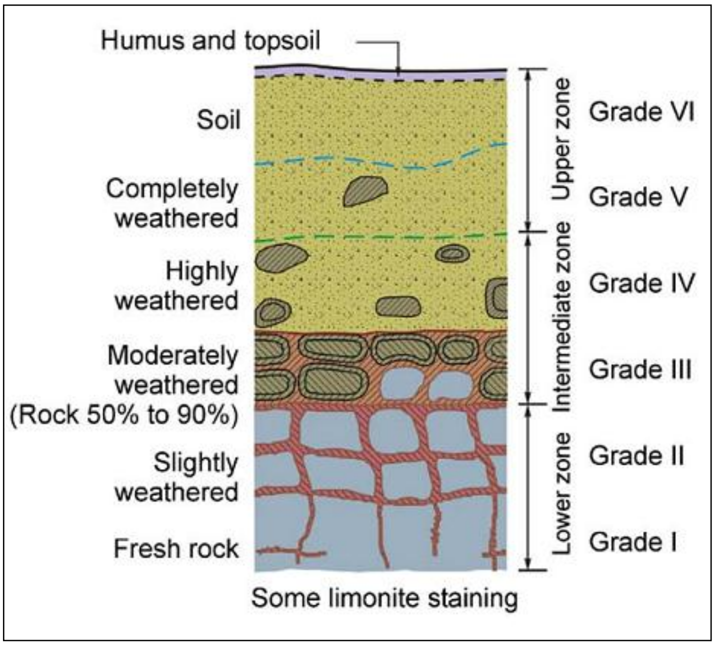

Figure 3.14. Typical weathering profile of residual soil (Little , 1969)

Undisturbed soil sample is collected at the interface level between bed rock and residual soil (Stage V – VI) for testing find the shear strength parameter (cohesion and friction angle) including consolidation drained test (8 test pits included TP 1 - TP 8) and consolidation undrained test (1 test pits included TP 9) with location of collected undisturbed soil sample in study area (see figure 3.15) Furthermore, collected the disturbed soil sample(11 samples included BNP 1 - BNP 11) for testing find the physical properties and soil permeability test lead to Geotechnical engineering laboratory with position of collected Disturbed soil sample in study area (see figure 3.16). Disturbed soil samples are likely to be more representative in the area. Disturbed samples are often taken for moisture content or plasticity determination in the laboratory, and in association with determinations of in situ density.

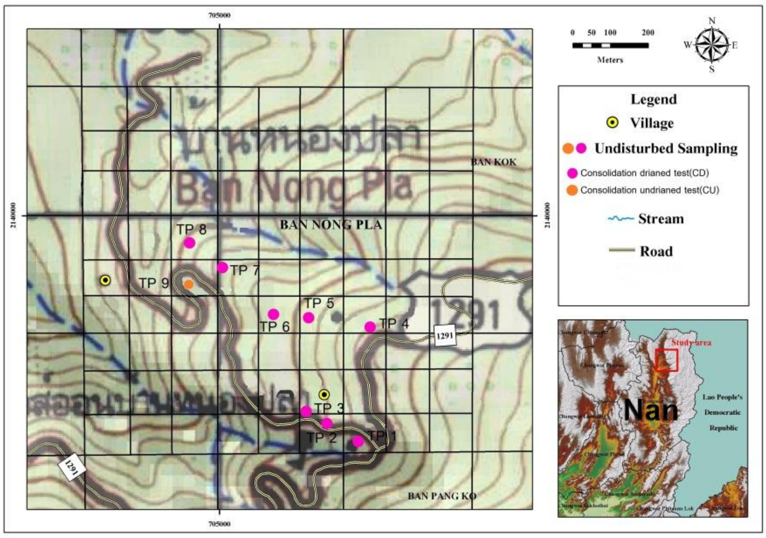

Figure 3.15. location of collected undisturbed soil sample in study area.

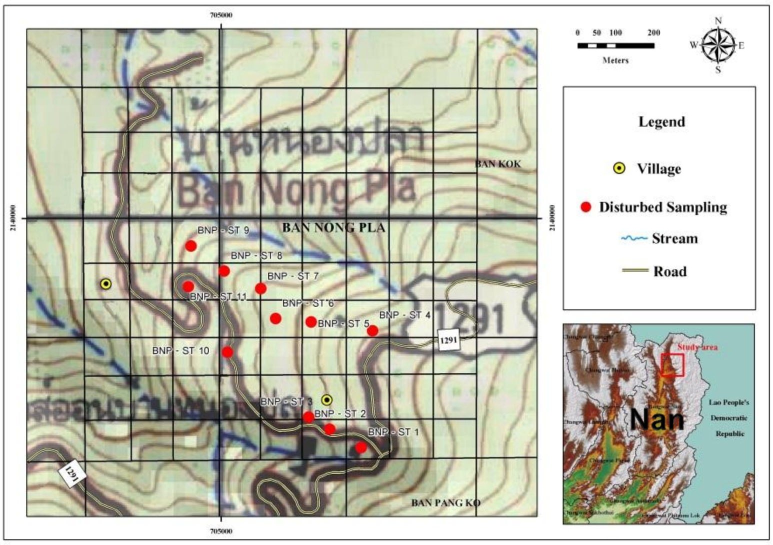

Figure 3.16. location of collected Disturbed soil sample in study area.

3.7 Physical properties and engineering properties of the soils

This research will focus on analyze engineering properties and physical properties of residual soil from weathered parent rock and collected residual soil from study area for testing in Geotechnical engineering laboratory included grain size distribution, moisture content, total unit weight specific gravity, porosity, void ratio, permeability, consistency for residual soils and shear strength parameter(cohesion of soil and friction angle). Furthermore, Study the relationship between shear strength with moisture content behavior of residual soils. The shear strength parameter used to analyze the stability of soil slopes. In order to find the plane of failure and find the safety factor of the soil slope.

Finally, The physical properties and engineering properties of residual soil samples were analyzed in geotechnical engineering laboratory by use the testing method of American Society for Testing and Materials (ASTM) including Total unit weight , Water content test(ASTM D 2216), Specific gravity test (ASTM D854), Sieve analysis test (ASTM D422) , Hydrometer test (ASTM D422), Atterberg's limits test (ASTM D4318), Permeability test (ASTM D 2434-68) and Direct shear test (ASTM D 3080).

3.8 Grain size analysis (ASTM D422)

Grain size analysis, which is among the oldest of soil test, is widely used in engineering classifications of soils. Grain-size analysis is also utilized in part of the specifications of soil for airfields, roads, earthdams, and other soil embankment construction. Additionally, frost susceptibility of soils can be fairly accurately predicted from the results of grain-size analysis. The standard grain-size analysis test determines the relative proportions of different grain sizes as they are distributed among certain size range. The test procedure which should be followed depends on the soil in question. If nearly all its grains are so large that they cannot pass through square openings of 0.074 mm (No.200 screen), the sieve analysis is preferable. For those soils which are nearly all finer than a No.200 screen, the hydrometer test is recommended. For silts, silty clays, etc., which have a measurable portion of their grains both coarser and finer than No.200 sieve, the combined analysis is needed. Grain-size analysis helps to classify soils, especially coarse soils. It is possible to tell from grain size distribution analysis whether the soil consists of predominantly gravel, sand, silt, or clay, and to a limited extent, which of these size ranges is likely to control the soil engineering properties.

Grain-size analysis is of greater value if supplemented by descriptive details such as color and particle shape. But the engineering behavior of soils also depends on factors other than particle sizes, such as mineral, structural, and geological history. The physical behavior of clays, such as plastic consistency, controls more of its mechanical behavior than its particle size distribution, and for this the Atterberg' s limits test provides more significant information than is provided by grain-size analysis.

3.8.1 Sieve analysis

Sieving is to determine the grain size distribution curve of soil samples by passing them through a stack of sieves of decreasing mesh-opening sizes and by measuring the weight retained on each sieve. The sieve analysis is generally applied to the soil fraction larger than 75 μm Grains smaller than 75 μm are sorted by using sedimentation (e.g., hydrometer or pipette analysis). Sieving can be performed in either wet or dry conditions. Dry sieving is used only for soils with a negligible amount of plastic fines, such as gravels and clean sands, whereas wet sieving is applied to soils with plastic fines. Grain size distribution curves enable sands and gravels to be classified into three main types: uniform, well graded, and poorly graded.

3.8.2 Hydrometer analysis

Hydrometer analysis is used to determine the grain size distribution of fine-grained soils having particle sizes smaller than 75 μm. The principle of hydrometer analysis is based on Strokes's law. It assumes that dispersed soil particles of various shapes and size fall in water under their own weight as non-interacting spheres. The grain size is calculated from the distance of sedimentation of soil particles. The percent by weight finer is determined by measuring the unit weight of the soil-fluid suspension.

3.9 Specific Gravity (ASTM D854)

In general, the term specific gravity of soil is the ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at temperature of 4 ^{o}C. In effect, it tells how much the material is heavier (or lighter) than water. Although specific gravity is employed in identification of naturals, it is of limited value for identification or classification of soils because the specific gravities of most soils fall within a narrow range. The range of the values of specific gravity is narrowed, typically between 2.65 - 2.78, thus, it is the necessity to carry the test with precautions.

The specific gravity of soil is used in calculating the phase relationships of soils (that is, the relative volumes of solids to water and air in a given volume of soil). The specific gravity (G_s) can be used for computing void ratio, total density, and dry density.

3.10 Atterberg' s Limits (ASTM D4318)

The engineering behavior of fine-grained soils depends on factors other than particle size distribution and density. It is influenced primarily by their mineral and structural composition and the amount of water they contain, which is referred to as water content. The liquid and plastic limits tests characterize the effects of water content on fined-grained soils on their consistency (liquid, plastic, semi-solid and solid) in disturbed condition and help to classify fine-grained soils and to assess their mineral composition and engineering properties. A fine-grained soil can exist in any of several states; which state depends on the amount of water in the soil system. When water is added to dry soil, each particle is covered with a film of absorbed and double layer water. If the addition of water is continued, the thickness of the water film on a particle increases. Increasing the thickness of the water films permits the particles to slide past one another more easily. The behavior of the saturated fine-grained soil, therefore, is related to the amount of water in the system.

3.10.1 Liquid and Plastic Limits

Approximately forty years ago, A.Atterberg defined the boundaries of four states of saturated soils in terms of “limits” as follow: (a) “Liquid limit, ” this water content is referred to be the boundary between the liquid and plastic states, is the minimum water content at which the disturbed fine grained soil will behave like viscous liquid (i.e. flow easily) and at this liquid limit water content, any soil will have the undrained shear strength about 2.0 kPa; (b) “Plastic limit,” is the minimum water content at which the saturated soil can remained to be in plastic state. At this plastic condition the saturated soil will deform at constant volume. This water content is, therefore, the boundary between the plastic and semi-solid states; and (c) “Shrinkage state,” is the minimum water content at which the soil has the minimum volume and remains to be saturated. Upon further drying, the unsaturated soil will have no volume change. This water content is referred to be the boundary between the semi-solid and solid states. These four states are show in figure 3.17

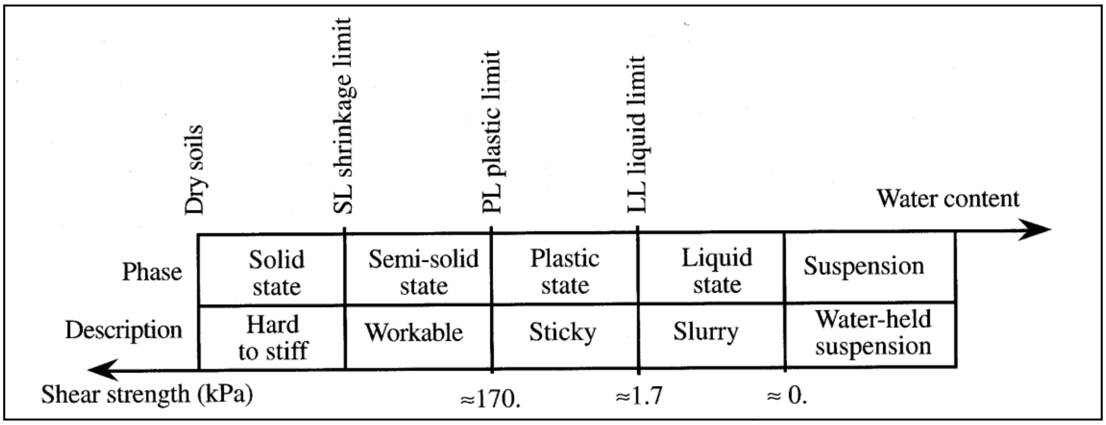

Figure 3.17. Variation of consistency of fine-grained soils with water content(Evett and Lui , 1997)

3.10.2 Fine - Grained Soil Classification

Fine-grained soils are usually classified by using the plasticity chart (figure 3.17). The plasticity chart is a graphical plot of the liquid limit w_L against the plasticity index I_p. The standard plasticity chart is shown in figure 3.18. When the values of w_L and I_p for inorganic clays are plotted on this chart, most of point lie just above the line marked *A-Line*. The *A-Line* is defined by the relationship:

$$I_p = 0.73(w_L-20)$$

Where I_p and w_L are in percents. The *A-line* is a reference line derived from experimental observations. It does not represent a well-defined boundary between soil types. The *U-Line* of figure 3.17 is a tentative upper limit for all soils, which was also drawn from experimental data, The *U-Line* has the equation:

$$I_p = 0.9(w_L-8)$$

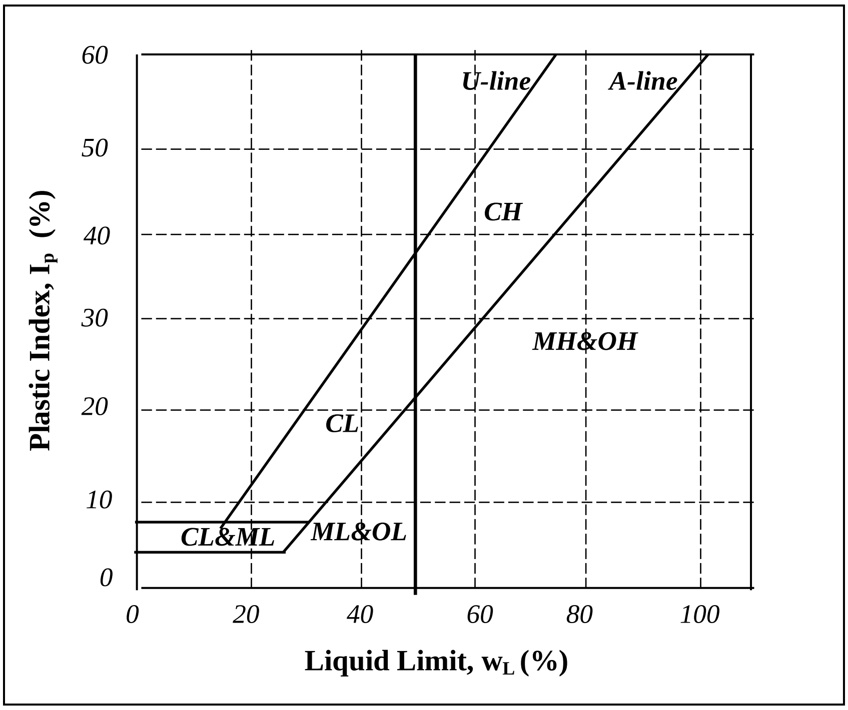

Figure 3.18. Plasticity chart for fine-grained soil classification (Evett and Lui , 1997)

In the above Figure 3.18, the notations C and M one referred for clay and silt respectively. The notations H and L are referred as low and high plasticity respectively. The letter O indicates the organic fine-grained soils of high or low plasticity (OH or OL).

Table 3.5 Typical values of Atterberg' s limits for clay minerals (Mitchell , 1976)

Mineral	Liquid Limit	Plastic Limit	Shrinkage Limit
Montmorillonite	100-900	50-100	8.5-15
Notronite	37-72	19-27	
Illite	60-120	35-60	15-17
Kaolinite	30-110	25-40	25-29
Hydrated halloysite	50-70	47-60	
Dehydrated halloysite	35-55	30-45	
Attapulgite	160-230	100-120	
Chlorite	44-47	36-40	
Allophane	200-250	130-140	

For soft Bangkok clay, the ranges of natural water content, plasticity index, and liquidity index are between 70 - 130 %, 30 - 80 %, and 0.7 - 1.3 respectively.

3.11 Permeability Test (ASTM D2434)

Soils are permeable to water because the voids between soil particles are interconnected. The degree of permeability is characterized by the permeability coefficient *k*, also referred to as hydraulic conductivity. In the laboratory, *k* is measured by using either the constant head test for soils of high permeability (e.g., sands), or the falling head test for soils of intermediate and low permeability (e.g., silts and clays).

Permeability refers to the propensity of material to allow fluid to move through its pores or interstices. In the context of soil, permeability generally relates to the propensity of soil to allow water to move through its void spaces. According to Darcy's law, the laminar flow rate of water *q* through a saturated soil of cross-sectional area *A* is directly proportional to the imposed hydraulic gradient (slope) *i*, or

$$\frac{q}{A} \sim i$$

If a constant of proportionality *k* is introduced, we obtain the equation

$$q = kiA$$

The constant *k* is known as the coefficient of permeability, or just permeability. Obviously, it indicates the ease with which water will flow through a given soil. The greater the value of permeability, the greater the flow will be for a given area and gradient.

Permeability is an important soil parameter for any project where flow of water through soil is a matter of concern- for example, seepage through or under a dam and drainage from subgrades or backfills. There are several factors that mostly influence the permeability of a soil: the viscosity of its water (which is a function of temperature), size and shape of the soil particles, degree of saturation, and void ratio. The void ratio is a significant influence on permeability. For a given soil, permeability is inversely proportional to soil density. This is intuitively obvious if one considers that the denser the soil, the more tightly its particles are packed, the smaller will be the void space and void ratio, and the lower will be the tendency for the soil to allow water to move through it. Hence, permeability is directly proportional to void ratio. The coefficient of permeability has the same unit as velocity. The coefficient of permeability of soils is dependent on several factor included fluid viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of mineral particles, and degree of soil saturation(Das ,1994). In clayey soils, structure plays an important role in the coefficient of permeability. Other major factors that affect the permeability of clays are the ionic concentration and the thickness of layers of water held to the clay particles.

3.12 Classification of Soil

Different soils with similar properties may be classified into groups and subgroups based on their engineering behavior. Classification systems provide a common language to concisely express the general characteristics of soils, which are infinitely varied, without detailed descriptions. Most of the soil classification systems that have been developed for engineering purpose are based on simple index properties such as particle size distribution and plasticity. Although several classification systems are now in use, one is totally definitive of any soil for all possible applications because of the wide diversity of soil properties

The principal objective of any soil classification system is predicting the engineering properties and behavior of a soil based on a few simple laboratory or field tests. Laboratory and/or field test results are then used to identify the soil and put it into a group that has soils with similar engineering characteristics. Probably no existing classification system completely achieves the stated objective of classifying soils by engineering behavior because of the number of variables involved in soil behavior and the variety of soil problems encountered. Considerable progress has been made toward this goal, particularly in relationship to soil problems encountered in highway and airport engineering. Soil classification should not be regarded as an end in itself but as a tool to further your knowledge of soil behavior. The Unified soil classification system (USCS) is based on the characteristics of the soil that indicate how it will behave as a construction material.

3.12.1 Unified Soil Classification System

The original form of this system was proposed by Casagrande in 1942 for use in the air field construction works undertaken by Army Corps of Engineers during World War II. In cooperation with the U.S. Bureau of Reclamation, this system was revised in 1952. At present, it is widely used by engineers (ASTM designation D 2487). The Unified classification system is presented in Tables 3.6, 3.7 and 3.8. This system classifies soils into two broad categories:

1. Coarse grained soils that are gravelly and sandy in nature with less than 50 % passing through the No. 200 sieve. The group symbols start with prefixes of either G or S. G stands for gravel or gravelly soil, and S for sand or sandy soil.
2. Fine grained soils with 50 % or more passing through the No.200 sieve. The group symbols start with prefixes of M, which stands for silt or inorganic silt, C for clay or inorganic clay, and O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils.

Other symbols used for the classification are

W ------- well graded

P -------- poorly graded

L -------- low plasticity (liquid limit less than 50)

H -------- high plasticity (liquid limit more than 50)

Table 3.6 Unified Classification System - Group Symbol for Gravelly soil (Das,1994)

Group symbol	Criteria
GW	Less than 5% passing No.200 sieve ; $C_u = D_{60}/ D_{30}$ greater than or equal to 4 ; $C_c = (D_{30})^2 / (D_{10} \times D_{60})$ between 1 and 3
GP	Less than 5% passing No.200 sieve ; not meeting both criteria for GW
GM	More than 12% passing No.200 sieve ; Atterberg's limits plot below A-line (figure 3.19) or plasticity index less than 4
GC	More than 12% passing No.200 sieve ; Atterberg's limits plot above A-line (figure 3.19) or plasticity index less than 7
GC - GM	More than 12% passing No.200 sieve ; Atterberg's limits fall in hatched area marked CL- ML in figure 3.19
GW - GM	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for GW and GM
GW - GC	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for GW and GC
GP - GM	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for GP and GM
GP - GC	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for GP and GC

Table 3.7 Unified Classification System - Group Symbol for Sandy soil (Das,1994)

Group symbol	Criteria
SW	Less than 5% passing No.200 sieve ; $C_u = D_{60}/ D_{30}$ greater than or equal to 6 ; $C_c = (D_{30})^2 / (D_{10} \times D_{60})$ between 1 and 3
SP	Less than 5% passing No.200 sieve ; not meeting both criteria for SW
SM	More than 12% passing No.200 sieve ; Atterberg's limits plot below A-line (figure 3.19) or plasticity index less than 4
SC	More than 12% passing No.200 sieve ; Atterberg's limits plot above A-line (figure 3.19) or plasticity index less than 7
SC - SM	More than 12% passing No.200 sieve ; Atterberg's limits fall in hatched area marked CL- ML in figure 3.19
SW - SM	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for SW and SM
SW - SC	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for SW and SC
SP - SM	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for SP and SM
SP - SC	Percent passing No.200 sieve is 5 to 12 ; meets the criteria for SP and SC

Table 3.8 Unified Classification System - Group Symbol for Silty and Clayey soils (Das, 1994)

Group symbol	Criteria
CL	Inorganic ; LL< 50 ; PI < 7 ; plot on or above A-line (see CL zone in figure 3.19)
ML	Inorganic ; LL< 50 ; PI < 4 ; plot on or below A-line (see ML zone in figure 3.19)
OL	Organic ; (LL – oven-dried) / (LL - not dried) < 0.75 ; LL < 50 (see OL zone in figure 3.19)
CH	Inorganic ; LL $\geq$ 50 ; PI plot on or above A-line (see CH zone in figure 3.19)
MH	Inorganic ; LL $\geq$ 50 ; PI plot on or below A-line (see MH zone in figure 3.19)
OH	Organic ; (LL – oven-dried) / (LL - not dried) < 0.75 ; LL $\geq$ 50 (see OH zone in figure 3.19)
CL - ML	Inorganic ; plot in the hatched zone in figure 3.19
Pt	Peat , muck, and other highly organic soils

For proper classification according to this system, some or all of the following need to be known :

1. Percent of gravel ---- that is, the fraction passing the 3-in. sieve (76.2-mm opening) and retained on the No.4 sieve (4.75-mm opening)
2. Percent of sand ---- that is, the fraction passing the No.4 sieve (4.75-mm opening) and retained on the No.200 sieve (0.075-mm opening)
3. Percent of silt and clay ---- that is, the fraction finer than the No.200 sieve (0.075 mm opening)
4. Uniformity coefficient (C_u) and the coefficient of gradation (C_c)
5. Liquid limit and plasticity index of the portion of soil passing the No.40 sieve

The group symbols for coarse-grained gravelly soils are GW,GP, GM,GC ,GC-GM, GW-GM, GW-GC ,GP-GM, and GP-GC. Similarly, the group symbols for fine-grained soils are CL, ML , OL ,CH, MH , OH, CL-ML, and Pt.

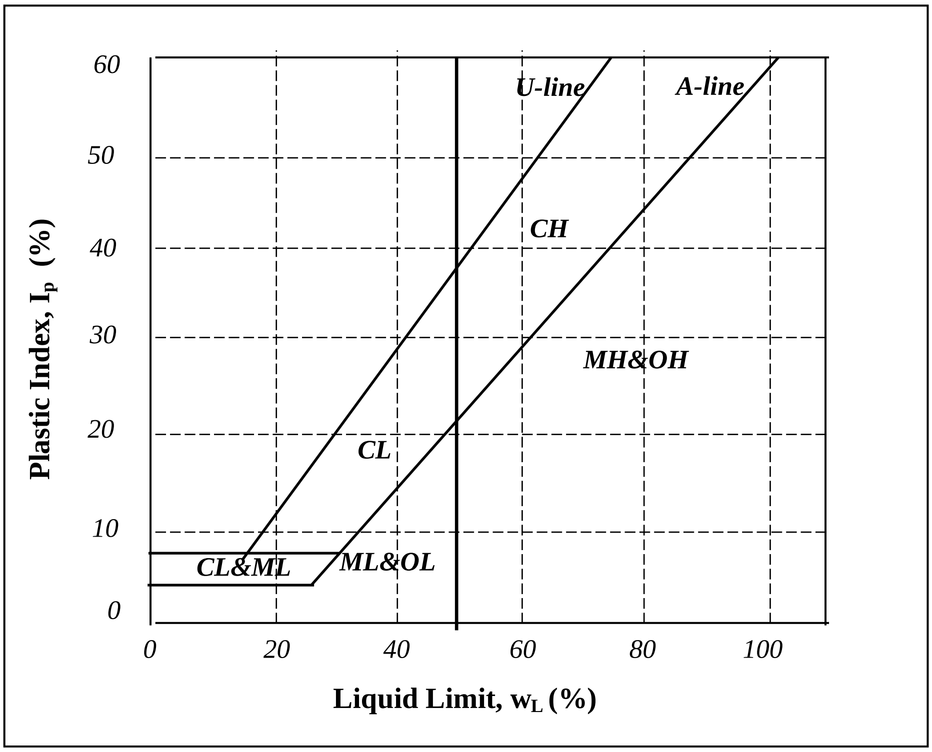

Figure 3.19. Plastic chart (Evett and Lui , 1997)

3.13 Shear strength of soil

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it (Das ,1994). One must understand the nature of shearing resistance in order to analyze soil stability problems such as bearing capacity, slope stability , and lateral pressure on earth-retain structure.

3.13.1 Mohr- coulomb failure criteria

Mohr(1900) presented a theory for rupture in materials that a material fails because of a critical combination of normal stress and shearing stress, and not from either maxi mum normal or shear stress alone. Thus, the functional relationship between normal stress and shear stress on failure plane can be expressed in the form figure 3.20.

$$\tau_f = f(\sigma) \tag{1}$$

The failure envelope defined by Equation(1) is a curved line, as shown in figure 3.20 For most soil mechanics problems, it is sufficient to approximate the shear stress on the failure plane as a linear function of the normal stress (Coulomb,1776). This can be written as

$$\tau_f = c + \sigma \tan\phi \qquad (2)$$

Where τ_f = shear strength of the soil

c = cohesion

σ = normal stress on the plane of failure

ϕ = angle of internal friction

The preceding relationship is called the Mohr-Coulomb failure criteria.

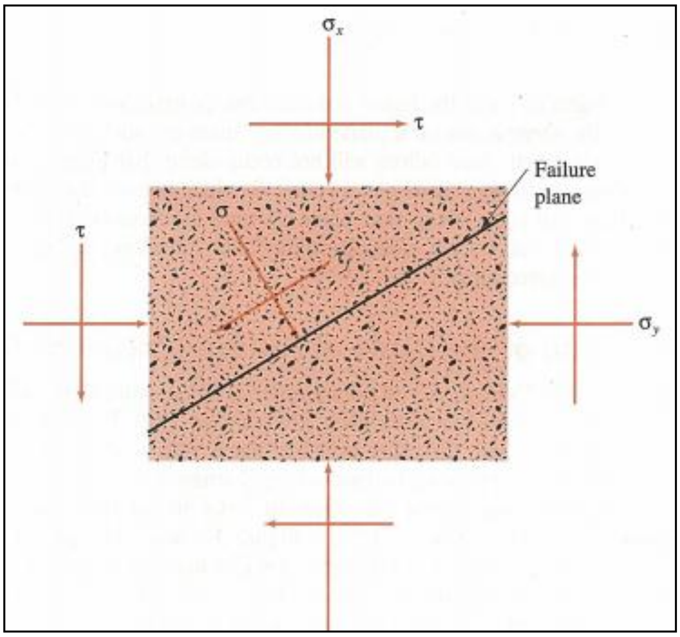

Figure 3.20. Relationship between normal stress and shear stress on a failure plane (Das ,1994).

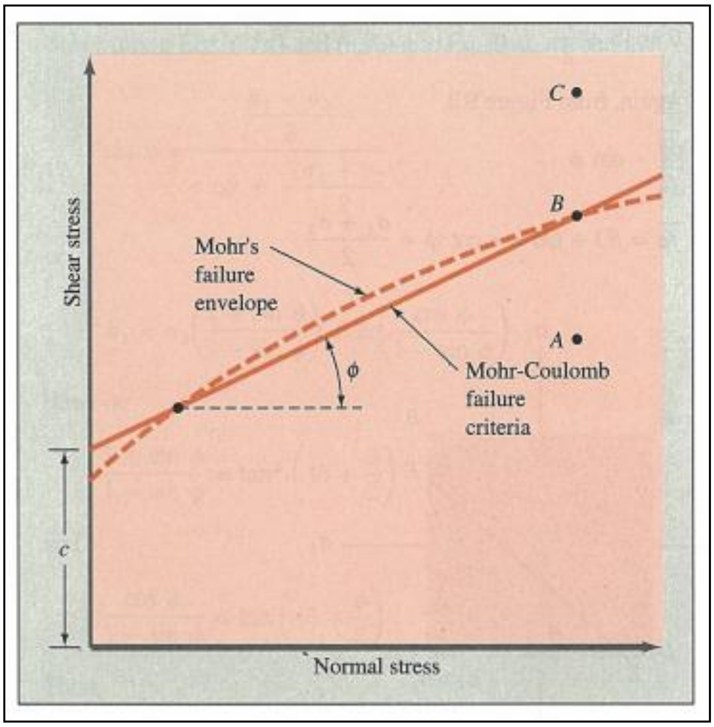

Figure 3.21. Mohr's failure envelope and Mohr- Coulomb failure criteria (Das ,1994).

The significance of the failure envelope can be explained as follows. If the normal stress and the shear stress on a plane in a soil mass are such that they plot as point A in figure 3.21, then shear failure will not occur along that plane. If the normal stress and the shear stress on a plane plot as point B (which falls on the failure envelope), then shear failure will occur along that plane. A state of stress on a plane represented by point C cannot exist, since it plots above the failure envelope, and shear failure in a soil would have occurred already.

3.14 Determination of shear strength parameters for soils in the laboratory

The shear strength parameters of a soil can be determined in the laboratory by primary two type of test : direct shear test and triaxial test. The procedure for conducting each of these tests are explained in some detail in the following sections.

3.14.1 Direct shear test (D 3080)

This is the oldest and simplest from of shear test arrangement. A diagram of the direct shear test apparatus is shown in figure 3.22. The test equipment consists of metal shear box in which the soil specimen is placed. The soil specimens may be square or circular in plan. The size of the specimens generally used is about 3 or 4 in.(6.1 – 6.3mm) across and about 1 in.(25.4 mm) high. The box is spit horizontally into halves. Normal force on the specimen is applied from the top of shear box. The normal stress on the specimens can be as great as 150 lb/in^2 (1034.2 kN/m^2). Shear force is applied by moving one half of the box relative to cause failure in the soil specimen. Depending on the equipment, the shear test can be either stress-controlled or strain-controlled.

In stress-controlled tests, the shear force is applied in equal increments until the specimen fails. The failure takes place along the plane of split of the shear box. After the application of each incremental load, the shear displacement of the top half of the box is measured by a horizontal dial gauge. The change in the height of the specimen (and thus the volume change of the specimen) during the test can be obtained from the readings of a dial gauge that measures the vertical movement of the upper loading plate.

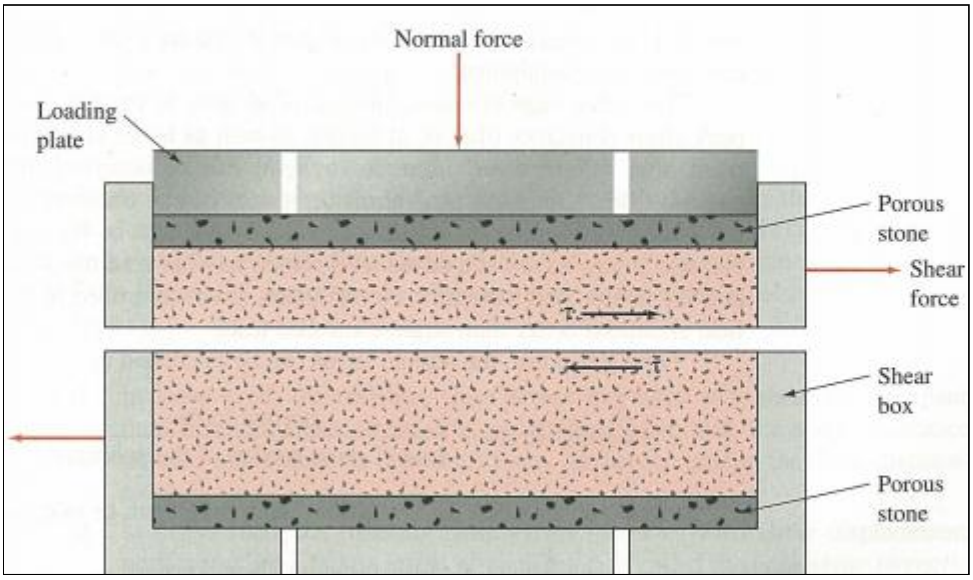

Figure 3.22. Diagram of direct shear test arrangement (Das ,1994)

In strain-controlled tests, a constant rate of shear displacement is applied to one half of the box by a motor that acts through gears. The constant rate of shear displacement is observed by a horizontal dial gauge. The resisting shear force of the soil corresponding to any shear displacement can be measured by a horizontal proving ring or load cell. The volume change of the specimen during the test is obtained in a manner similar to the stress-controlled tests.

The advantage of the strain-controlled tests is that, in the case of dense sand, peak shear resistance (that is, at failure) as well as lessor shear resistance(that is, at a point after failure called *ultimate strength*) can be observed and plotted. In stress-controlled tests, only the peak shear resistance can be observed and plotted. Note that the peak shear resistance in stress-controlled tests can be only approximated. This is because failure occurs at a stress level somewhere between the prefailure load increment and the failure load increment. Nevertheless, stress-controlled tests probably model real field situation better than strain-controlled tests. For a given test, the normal stress can be calculated as

$$\sigma = \text{Normal stress} = \frac{\text{normal force}}{\text{Area of cross-section of the specimen}}$$

The resisting shear stress for any shear displacement can be calculated as

$$\tau = \text{Shear stress} = \frac{\text{resisting shear force}}{\text{Area of cross-section of the specimen}}$$

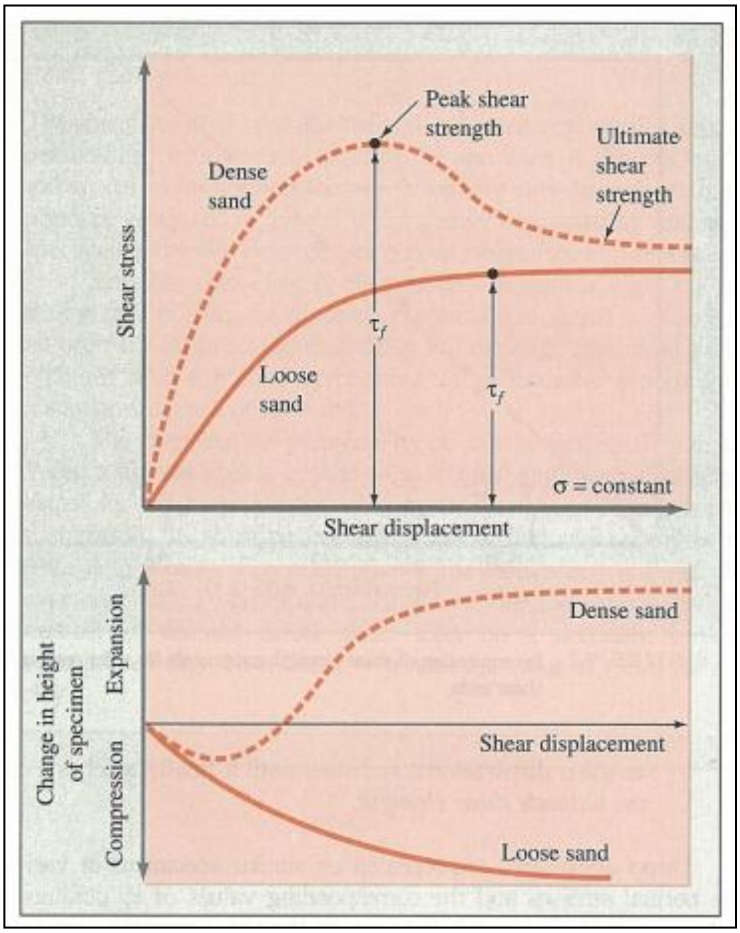

Figure 3.23. Plot of shear stress and change in height of specimen against shear displacement for loose and dense dry sand : direct shear test (Das ,1994)

from figure 3.23. shows a typical plot of shear stress and change in the height of the specimen against shear displacement for loose and dense sands. These observations were obtained from a strain-controlled test. The following generalizations can be developed from figure 3.23 regarding the variation of resisting shear stress with shear displacement :

1. In loose sand, the resisting shear stress increases with shear displacement until a failure shear of τ_f is reached. After that, the shear resistance remains approximately constant for any further increase in the shear displacement (Das ,1994).

2. In dense sand, the resisting shear stress increases with shear displacement until it reaches a failure stress of τ_f. This τ_f is call the *peak shear strength.* After failure stress is attained, the resisting shear stress gradually decreases as shear displacement increases until it finally reaches a constant value called the *ultimate shear strength* (Das ,1994).

3.14.1.1 Consolidation undrained test

This test is also called the quick test and performed with the drain valve closed for all phases of the test(water is not allowed drain) . The *undrained test* may be used to obtain shear strength parameters of clays and sand clay mixes under total stress conditions. Effective stresses cannot be measured since pore pressures generated during shearing cannot be measured. To minimize drainage, the normal loads should be kept low and shearing should be commenced as soon as possible after the application of the normal load. When failure is reached there will often be a peak value followed by a lower constant value. The test should continue until the shear force is constant and both the peak and lower value should be reported. As is the case with the undrained test only shear strength parameters at total stress conditions can be measured. Complete prevention of drainage is not possible as porous plates are used to allow consolidation. Drainage can be minimized by rapid shearing. Three hours is considered to be a conservative time for primary consolidation to take place in a 25 mm specimen for most clays, but this should be monitored for each specimen.

3.14.1.2 Consolidation drained test

The shear box that contains the soil specimen is generally kept inside a container that can be filled with water to saturate the specimen. *A drained test* is made on a saturated soil specimen by keeping the rate of loading slow enough so that the excess pore water pressure generated in the soil is completely dissipated by drainage. Pore water from the specimen is drained through two porous stones (see figure 3.22). Since the coefficient of permeability of sand is high, the excess pore water pressure. Since the coefficient of permeability sand is high, the excess pore water pressure generated because of loading (normal and shear) is dissipated quickly. Hence, for an ordinary loading rate, essentially full drainage conditions exist.

The friction angle (ϕ) obtained from a drained direct shear test of saturated sand will be the same as that for a similar specimen of dry sand. The coefficient of permeability of clay is very small compared with that of sand. When a normal load is applied to a clay soil specimen, a sufficient length of time must elapse for full consolidation that is, for dissipation of excess pore water pressure. For that reason, the shearing load has to be applied at very slow rate.The drained test with put the loading for compress to loosening the water and then shear specimen with shearing rate is very slow by calculated from the rate of loosening the water.

$$t_f = 12.7 \times t_{100} \tag{3}$$

where t_f = rate of shearing specimen

t_{100} = time it takes a loosening water until it not occur collapsed (Head, 1982)

and

$$t_f = 50\ t_{50} \tag{4}$$

where t_{50} = time it takes a loosening water 50 % of collapsed (ASTM 3080)

Cheung et al. (1988) found the rate of shearing in the range between 0.007 – 0.6 mm/sec. The values of strength obtained are not different. Ho and Fredlund (1982) proposed apply the rate of test from consolidated at 95 % and the normal rate of shearing at about 0.001 % Strain per minute.

In general, the direct shear test is suited to the relatively rapid determination of consolidated drained strength properties because the drainage paths through the test specimen are short, thereby allowing excess pore pressure to be dissipated more rapidly than with other drained stress tests. The test can be made on all soil materials and undisturbed, remolded or compacted materials. During the direct shear test, there is rotation of principal stresses, which may or may not model field conditions. Moreover, failure may not occur on the weak plane since failure is forced to occur on or near a horizontal plane at the middle of the specimen. The fixed location of the plane in the test can be an advantage in determining the shear resistance along recognizable weak planes within the soil material and for testing interfaces between dissimilar materials.

3.14.2 Multi-stage direct shear test

The study to find shear strength of residual soils from study area for this research by have selected testing is called " Multi-stage direct shear test ". The testing is done by comparing the effective shear strength of residual soil in natural moisture content condition and varies degree of saturation, S_r in 3 levels: 60%, 80% and 100% or KU-MDS test (Soralump and Thowiwat, 2009). In the general direct shear test will use a soil sample in testing at least three sample (conventional test) for find the relationship in type " Mohr - Coulomb failure envelope " to get the value of shear strength parameter but have the testing method can be performed find shear strength.

Taylor (1950) and Fleming (1952) success in the lead multi-stage test find the values of shear strength parameter (cohesion and friction angle) of unsaturated soil. Kenny and Watson (1961) testing Triaxial multi-stage type of saturated soil compare conventional test found the values of shear strength parameter is not different. (Kubtitaubhe, 1989) testing soil shear strength from Huai pra taw dam found Triaxial conventional test give the values higher than multi-stage test average of about 10% and compare testing Direct shear with Triaxial found the values of shear strength parameter of Direct shear test give the values lower.

(Kubtitaubhe, 1989) found important aspect in the Multi-stage direct shear testing that is point conversion the confining pressure. Therefore, must be consider " failure criteria " many respect combine that is

1. The failure can be observed such as the shearing plane in the specimen.
2. The Maximum Deviator Stress appears flat of the graph between Stress and Strain.

3. Appear the displacement as far as the strain values defined for plastic soil using the Strain at 16 %, 18 % and 20 % for ranges 1, 2 and 3.

4. Principal Stress Ratio $\frac{\sigma'_1}{\sigma'_3}$ in CU test for soil category such as Overconsolidated. The values of $\frac{\sigma'_1}{\sigma'_3}$ has be valuable highest before appear the highest stress.

5. Excess Pore Pressure has be valuable highest similar when $\frac{\sigma'_1}{\sigma'_3}$ has be valuable highest.

6. Consideration the Effective Stress Path (see figure 3.24) Watson and Kenney (1961) write the graph between P$'$, q$'$ and drag " Failure Envelope " shows that in the ranges one the values $\frac{\sigma'_1}{\sigma'_3}$ appear highest at point **S** . The testing continue to the point **U**. By pathway of stress be in line with " Failure Envelope". When is like this in the ranges one (including the ranges other).

The testing can be stop at point **T** after that can be increased Confining Pressure for during the next, therefore, makes it unnecessary testing to Deformation. To provide to the highest stress values(Maximum Deviator Stress).

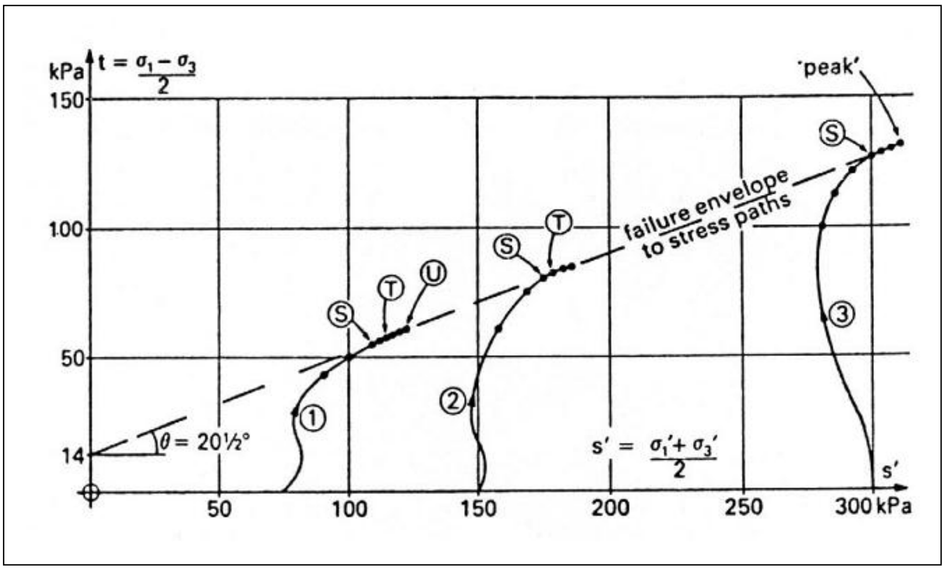

Figure 3.24.Graph of Effective stress from Multi-Stage Triaxial Test (Watson and Kenney, 1961)

Ho and Fredlund (1982) testing the Multi-Stage Triaxial test of Unsaturated soil found that The Multi-stage Cyclic Loading type (see figure 3.25) give result better than Multi-Stage Sustained Loading (see figure 3.26)because specimen remains in Elastic condition and not have force until appear “Creep “ during consolidated.

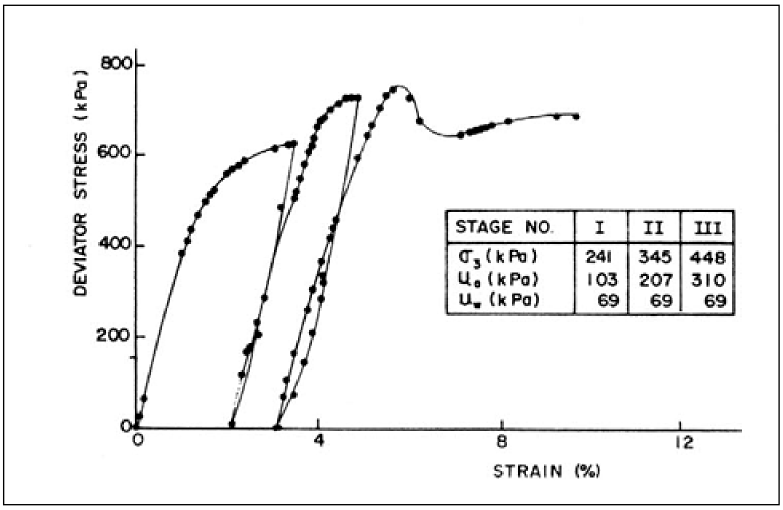

Figure 3.25. Stress - strain curve testing by Multi-Stage Cyclic loading Triaxial Test (Ho and Fredlund , 1982)

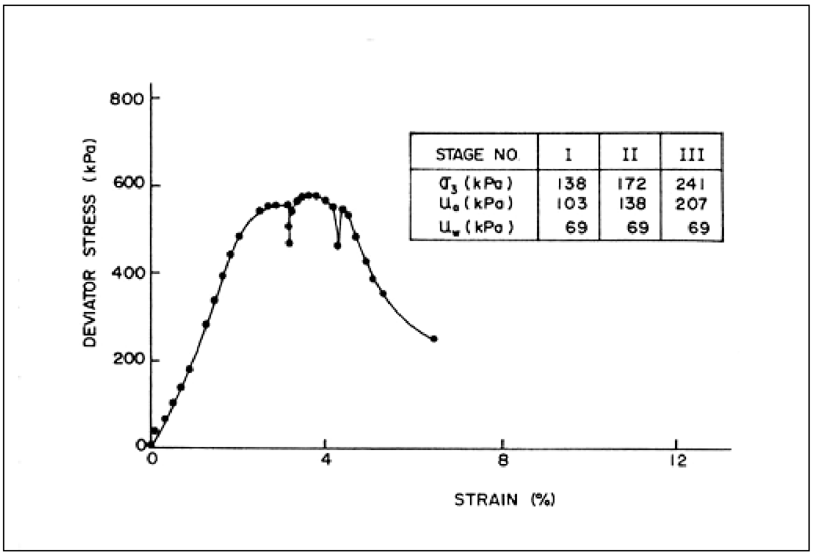

Figure 3.26. Stress – strain curve testing by Multi-Stage sustained loading Triaxial Test (Ho and Fredlund , 1982)

3.14.2.1 Application in geotechnical engineering

In general, the Multi-stage direct shear test is suited to the relatively rapid determination of consolidated drained strength properties and consolidated undrained strength properties. Test by shearing sample until nearly to the failure point in each normal load (at least three or four normal load). This testing method suitable for sample with high variability and in case have sample least, it can be used to this testing method. Which this testing method will give the value shear strength parameter reliable more than the conventional test.

When the soil are sampled directly from the actual or representative failure surfaces. The limited number of sample can be obtained so that the multi-stage direct shear test was used to minimize the testing sample. Only one soil sample with the specified moisture content was applied the initial normal stress close to existing overburden pressure. Then the sample was sheared until approach failure, the first stage was then stop. The higher normal stress was applied and repeat the shearing process similar to the first one. Then, the third or final stage were repeated the same as previous stages. The discrepancy was about 3-5 % on the conservative side between the conventional and multi-stage loading. The rate of shear should be on the order of *0.05 mm/min* for *consolidation drained test* and *0.1 mm/min for consolidation undrained test* (ASTM 3080)

3.15 Slope stability analysis

An exposed ground surface that stands at an angle with the horizontal is called an *unrestrained slope*. The slope can be natural or man-made. If the ground surface is not horizontal, a component of gravity will tend to move the soil downward as shown in figure 3.27 If the component of gravity is large enough, slope failure can be occur, that is, the soil mass in zone a, b, c, d, e, a can slide downward. The driving force overcomes the resistance from the shear strength of the soil along the rupture surface.

In many cases, civil engineer and engineering geologist are expected to check the safety of natural slopes, slopes of excavations, and compacted embankments. This check involves determining and comparing the shear stress developed along the most likely rupture surface with the shear strength of the soil. This process is called *slope stability analysis.* The stability analysis of a slope is not an easy task. Evaluation of variables such as the soil stratification and its in-place shear strength parameters may prove to be a formidable task. Seepage through the slope and the choice of a potential slip surface add to the complexity of the problem.

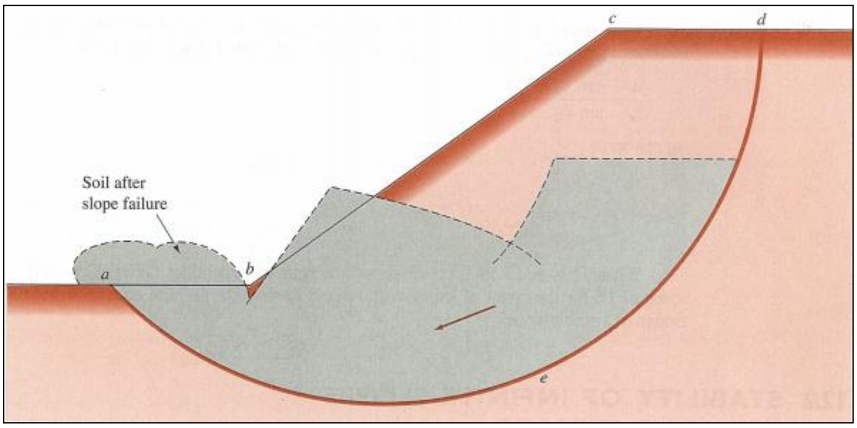

Figure 3.27. Slope failure (Das ,1994).

3.15.1 Factor of safety

The task of the engineer charged with analyzing slope stability is to determine the factor of safety. Generally, the factor of safety is defined as

$$F.S. = \frac{\tau_f}{\tau_d} = \frac{\text{Resisting Force}}{\text{Driving Force}} \qquad (5)$$

Where F.S. = Factor of safety

τ_f = Resisting Force

τ_d = Driving Force

The shear strength of a soil consists of two components, cohesion and friction, and may be written as

$$\tau_f = c + \sigma \tan\phi \tag{6}$$

Where τ_f = shear strength of the soil

c = cohesion

σ = normal stress on the plane of failure

ϕ = angle of internal friction

For disaster in a specific various such as surface motion is an arc of a circle, factor of safety refers to the ratio of the moment force around the center.

$$\text{F.S.} = \frac{M_R}{M_D} = \frac{\text{Moment of resistance force}}{\text{Moment of driving force}} \tag{7}$$

Where F.S. = Factor of safety

M_R = Moment of resistance to failure

M_D = Moment of driving to cause the failure

Typically, Step in the analysis of the stability of soil slopes "Limit Equilibrium" will be as follows.

1. Assumption or hypothesis the characteristic mode of failure. The surface movement collapse.
2. Calculation the resistance force adequate cause the balance of soil mass to failure.
3. Compare ratio between soil shear strength per effective resistance stress while balance which is called “Factor of Safety (F.S.)“.
4. By changing the characteristic or surface movement collapsed that is happen, or is likely to occur continuously to found the value ratio of safety factor (least). Should be characteristic failure likely to be occur (most) and find the factor of safety it should be.

3.15.2 Methods analyze the stability of the soil slope.

The analyze the stability of the soil slope done in several ways by method analyzing the common popular for example Method of Infinite Slope, Taylor Method , Ordinary Method of Slices , Simplified Bishop Method , Wedge Method. By the failure of soil slope has characteristic shape of surface motion is thin layer.

3.15.3 Infinite Slope analysis

Stability analysis of soil slope consistent with failure of soil slope that is " Method of Infinite slope " which looks the analysis appropriate with cause the failure soil slope because has a shape that looks is thin sheet movement down the surface slope and thickness of soil layer movement less than 1 in 10 of the length of the soil mass caused to failure. By considering that surface motion position parallel with surface of the slope, it may be dry soil and wet soil (has the flow of water parallel with surface slope). Which each case can be write " Free Body " and "Force Diagrams" (shown in figure 3.28). When considering the balance of forces parallel with surface movement.

Compare between Strength and Stress can will find the Factor of Safety from equation 8 to equation 11.

For ; Sand when $\tau = \sigma \tan\phi$ และ $\tau = \sigma \tan\phi$

Dry soil

$$F.S. = \frac{\tan\phi}{\tan\beta} \tag{8}$$

Wet soil (has the flow of water parallel with surface slope)

$$F.S. = \frac{\gamma_b}{\gamma_{sat}} \frac{\tan\phi}{\tan\beta} \tag{9}$$

For ; Clay when $\tau = c + \sigma \tan\phi$ and $\tau' = c' + (\sigma - u)\tan\phi$

Dry soil

$$F.S. = \frac{c}{\gamma \bullet h \sin\beta \bullet \cos\beta} + \frac{\tan\phi}{\tan\beta} \qquad (10)$$

Wet soil (has the flow of water parallel with surface slope)

$$F.S. = \frac{c'}{\gamma \bullet h \sin\beta \bullet \cos\beta} + \frac{\gamma_b}{\gamma_{sat}} \frac{\tan\phi'}{\tan\beta} \qquad (11)$$

A slope that extends for a relatively long distance and has a consistent subsoil profile may be analyzed as an infinite slope. The failure plane for this case parallel to the surface of the slope and the limit equilibrium method can be applied readily. A typical slice for a slope in dry sand is shown in figure 3.28 along with its free body diagram.

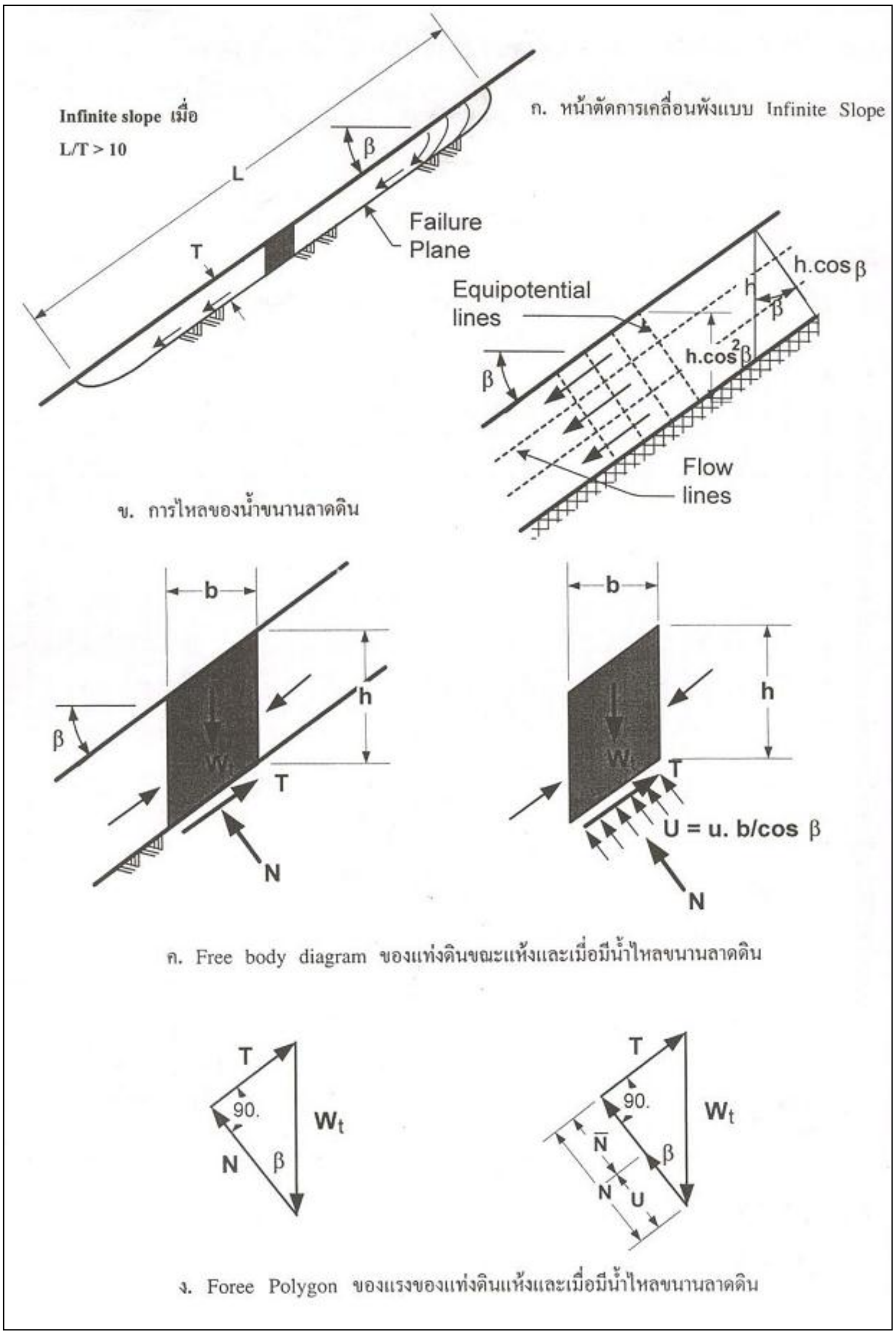

Figure 3.28. The shape of the surface of the soil mass movement collapsed in a thin sheet (Mairaing, 1999).

Chapter IV

RESULTS

4.1 Introduction

Chapter IV presents the results all about technical aspect of remote sensing and geographic information systems, 2-D electrical resistivity interpretation including method of multi-electrode resistivity imaging in the study area. In addition, test result of the physical properties of disturbed soil sample includes total unit weight, water content test (ASTM D 2216), specific gravity test (ASTM D 854), sieve analysis test (ASTM D 422), hydrometer test (ASTM D 422), Atterberg's limits test (ASTM 4318) and permeability properties of soil sample(recompacted material)(ASTM D 2434), classification of soil by unified classification system. Test result the engineering properties of undisturbed soil sample by Direct shear test (multi-stage type)(ASTM D 3080) show in this chapter includes shear strength parameter (cohesion and friction angle) and properties of soil shear strength decreases when the moisture content increases. Finally, slope stability analysis including Infinite slope analysis , KU- Slope Model calculated factor of safety of natural soil slope will be shown.

4.2 Interpretation satellite image

Remote sensing information is one of the most important data for landslide assessment. It is very useful in detecting and mapping landslide scars, land-use/land cover, upland agriculture, forest land (e.g., Akkrawintawong, 2008). Several types of remote sensing data are used to integrate landslide hazard assessment. The satellite images are used widely as fundamental knowledge for land planning, land use mapping, geological field survey, and landslide scar detection. This research applied data of the THEOS from Geo-Informatics and Space Technology Development Agency (GISTDA). The study area under investigation lies within the latitude 19° 26′ 39″ N, 19° 24′ 38″ N, 19° 13′ 29″ N, 19°11′ 35″ N and longitudes 100° 58′20″ E, 101° 11′ 34″ E, 100° 55′ 10 ″ E, 101° 08′ 24″ E

From THEOS satellite image interpretation, land use in this area can be classified as follows; area mostly agriculture such as rice plantation, lychee plantation, corn for animal feed plantation, urban and building up, forest land but not found landslide scar shown in figure 4.1. Direction of flow in the sub-watershed has shown the direction from the southeastern to northwestern flows into the Huai Nam Puea catchment. The development tends to move higher to the mountain. This changing of the land use and land cover may additionally induce landslide in the area. The consequences of population growth and demand for agricultural land are direct cause to alter the deforestation and land use and may trigger the erosion of soil leading to extensive landslide.

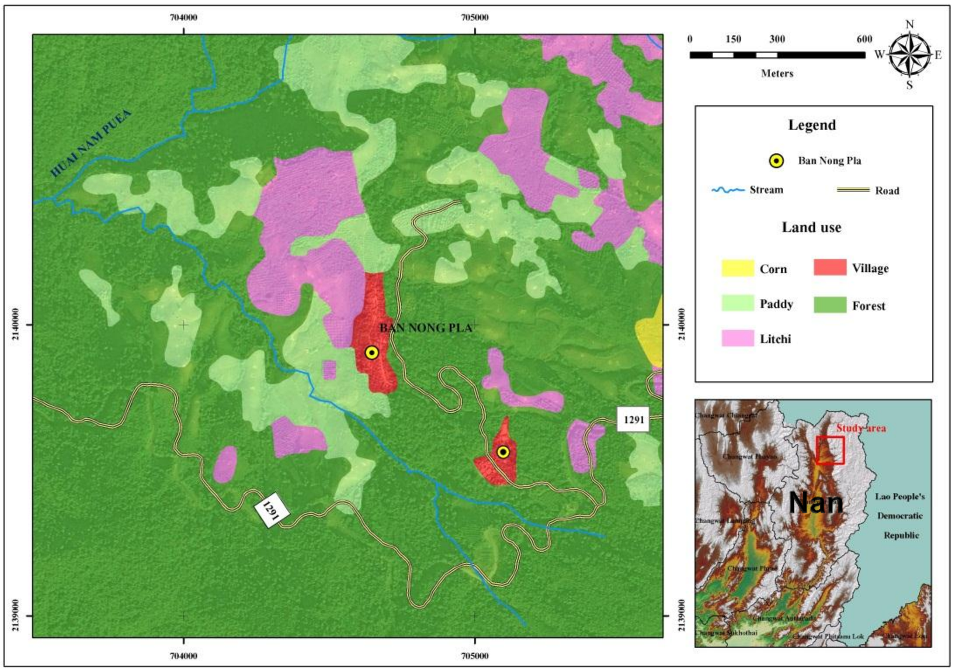

Figure 4.1. Results the satellite image interpretation.

4.3 Geographic Information system

Geographic information system is very useful tool for spatially distributed data processing and analysis. Conceptually, geographic information system should be able to utilize spatial data in any form, whether raster, vector or tabular. Geographic information system provides the following tasks of capabilities to handle geo-referencing data.

This research used geographic information system techniques for the specific mapping and classification different between landslide scars and land use area in the study area. The results using geographic information system are shown in figure 4.2. Topographic map overlaying basin can be divided Huai Nam Puea catchment into the 11 sub-catchments by first stream order. The study area, Ban Nong Pla, is located in two sub-catchments (with green color and blue color in figure 4.2).

In addition, from figure 4.3 stream order in Huai Nam Puea catchment can be divided into 4 stream orders including 1) first order (red color), 2) second order (orange color), 3) third order (light green color) 4) fifth order (dark green color), respectively.

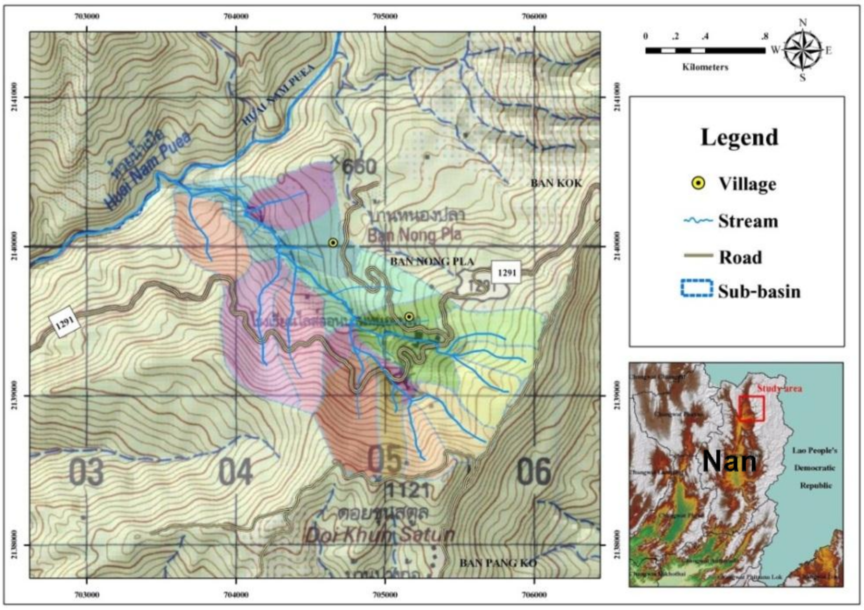

Figure 4.2. Topographic overlaying sub-catchment.

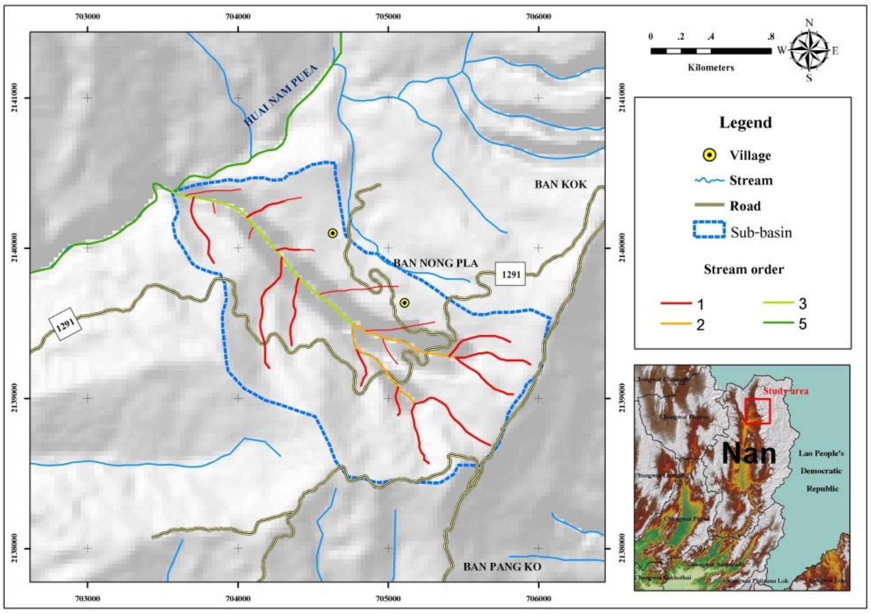

Figure 4.3. Stream order in Huai Nam Puea catchment.

In addition, from figure 4.5 The field investigation in study area found the clastic sedimentary rocks in Jurassic are a parent rock including shale, sandstone , mudstone and siltstone. These residual soil and weathered rocks have a rate of high weathering therefore resulting to high susceptibility for landslide.

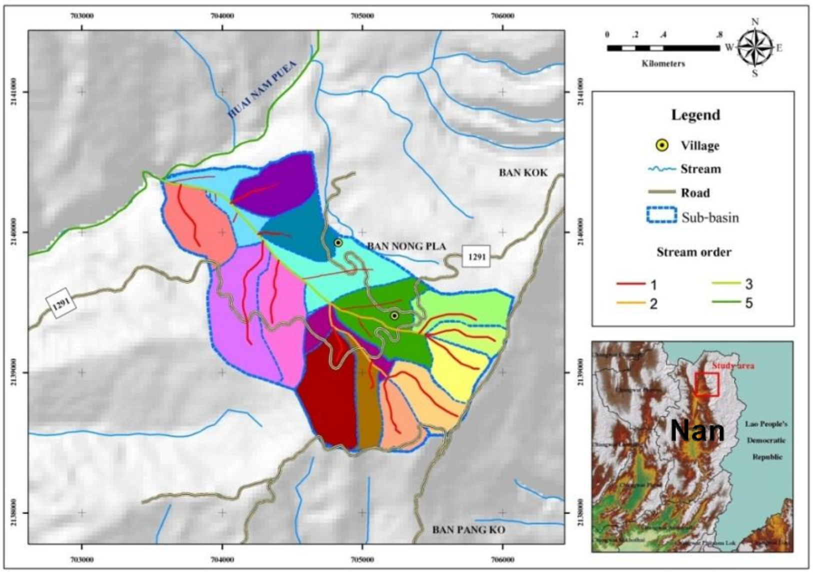

Figure 4.4. Sub-catchments overlaying stream order in Huai Nam Puea catchment.

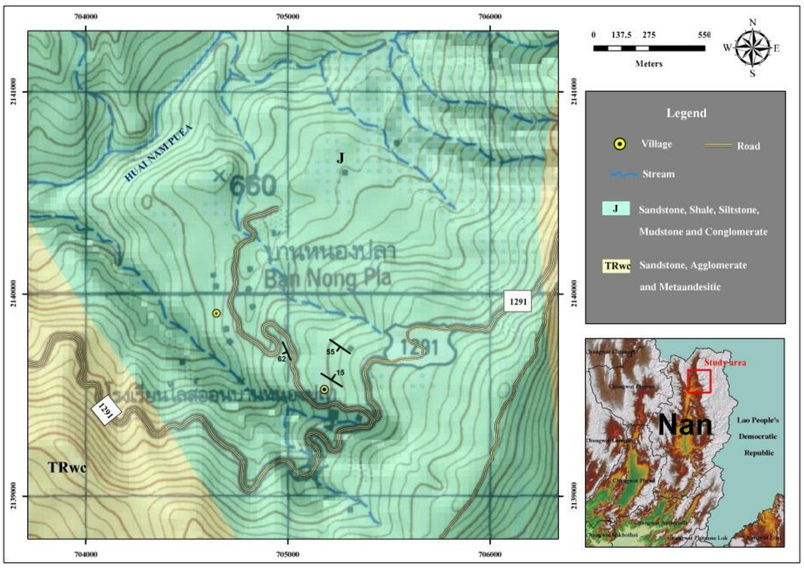

Figure 4.5. Characteristics of lithology type in study area (DMR , 2006).

4.4 2-D resistivity interpretation

2-D resistivity images obtained with such a multi-electrode technique are used for studying the shallow structures of the underground located a few tens of meters down to about one hundred meters depth. Several examples are presented for various types of applications, geotechnical (detection of a fault in a granitic area), archaeology (discovery of an ancient tomb) and mineral exploration (detection of a metallic ore body).

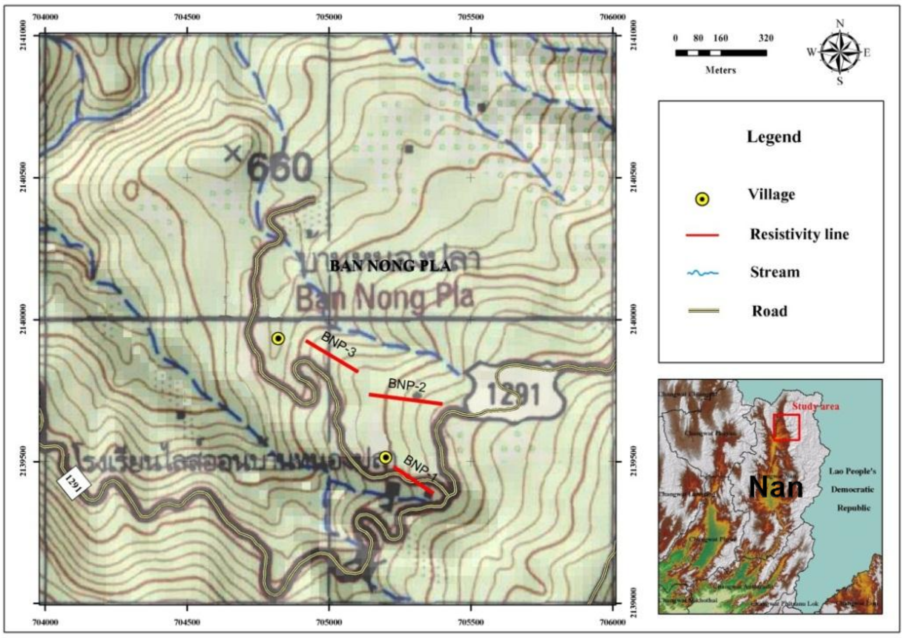

Figure 4.6. 2-D resistivity survey location in study area.

Survey lines in this research included BNP-1, BNP - 2, BNP-3 as shown in figure 4.6. BNP-1 is made from the northwestern to southeastern (NW – SE)direction, BNP – 2 is designed for eastern part to western part (E –W) and BNP – 3 is made from southeastern to northwestern part (SE – NW), respectively.

Table 4.1 Attribute of various zones based on resistivity values

Number	Zoning symbol	Description
1	a	Residual soil (Degree of weathering : Grade VI - IV), Resistivity values between 700 ohm.m - 1600 ohm.m
2	b	Shale slightly weathered and highly fractured shale, Resistivity values 400 ohm.m - 600 ohm.m
3	c	The fresh shale, Resistivity values 10 ohm.m - 300 ohm.m

Rock at resistivity line survey at BNP – 1 is fresh shale that is located in “zone **c**" has a resistivity values between 10 - 300 ohm.m (Nassereddine et al., 2011). This zone is characterized by *in situ* shale in the top and at the bottom of the profile, at a depth of 15 to 30 meters. Weathered and highly fractured shale represents "**zone b**" has a resistivity values between 400 – 600 ohm.m (Sowers, 1970), which encompasses *in situ* shale boulders in the middle and the bottom of profile. Finally, the "**zone a**" is of residual soil with resistivity values between 700 – 1600 ohm.m (Dorh, 1975; Griffiths and Barker, 1993; Jongman and Garambois, 2007). This zone is the result of the weathering process that change the rock into soil and has approximate thickness average values as 2- 5 meters(see figure 4.7).

Resistivity line survey at BNP – 2 is also characterized by fresh shale that is in “zone **c** " has a resistivity values between 10 - 300 ohm.m (Nassereddine et al., 2011). This zone is dominated by *in situ* shale in the profile, at the depth of 10 to 40 meters. Weathered and highly fractured shale is represented by in “zone **b**" that has a resistivity values between 400 – 600 ohm.m (Sowers, 1970) on which it encompasses *in situ* shale boulders in the top of profile. Finally, the "**zone a**" is of residual soil with resistivity values between 700 – 1600 ohm.m (Dorh, 1975; Griffiths and Barker, 1993; Jongman and Garambois, 2007). This zone is the result of the weathering process that change the rock into soil and has approximate thickness average values as 3- 9 meters(see figure 4.8).

Resistivity line survey at BNP – 3 shows relatively fresh shale that is in “zone **c**" has a resistivity values between 100 - 300 ohm.m (Nassereddine et al., 2011). This zone is characterized by *in situ* shale in the top and middle of the profile, at a depth of 1 to 25 meters. Weathered and highly fractured shale in “zone **b**" has a resistivity values between 400 ohm.m - 600 ohm.m (Sowers, 1970), which encompasses *in situ* shale boulders in the top of profile. Finally, the " **zone a**" is of residual soil with resistivity values between 700 - 1600 ohm.m (Dorh, 1975; Griffiths and Barker, 1993; Jongman and Garambois, 2007). This zone is the result of the weathering process that change the rock into soil and has approximate thickness average values as 2- 7 meters (see figure 4.9).

Lines survey BNP- 2 shows thickness of soil between 3 - 9 meters to which this range in thickness owns an opportunity to cause the landslide. During heavy rain, this soil will be more opportunity to slide than a normal situation because of fine-grained of residual soils. The increases in water volume in soil may result the decrease of strength in soil.

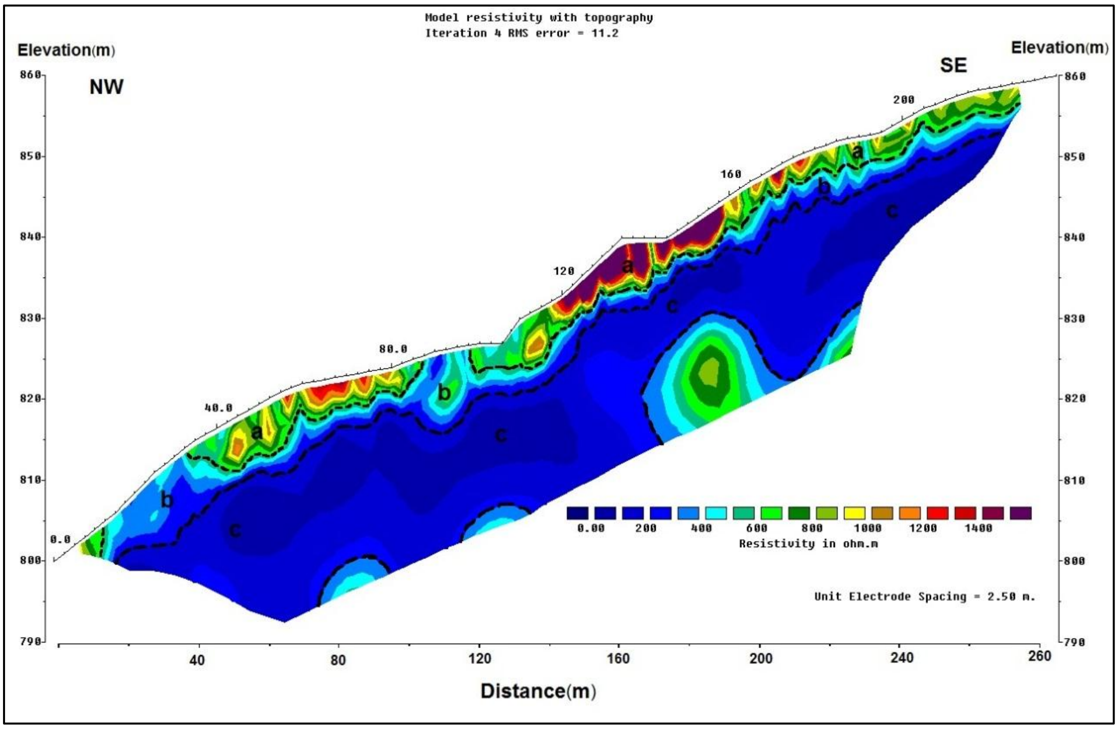

Figure 4.7. Result the resistivity with topography interpretation (BNP –1).

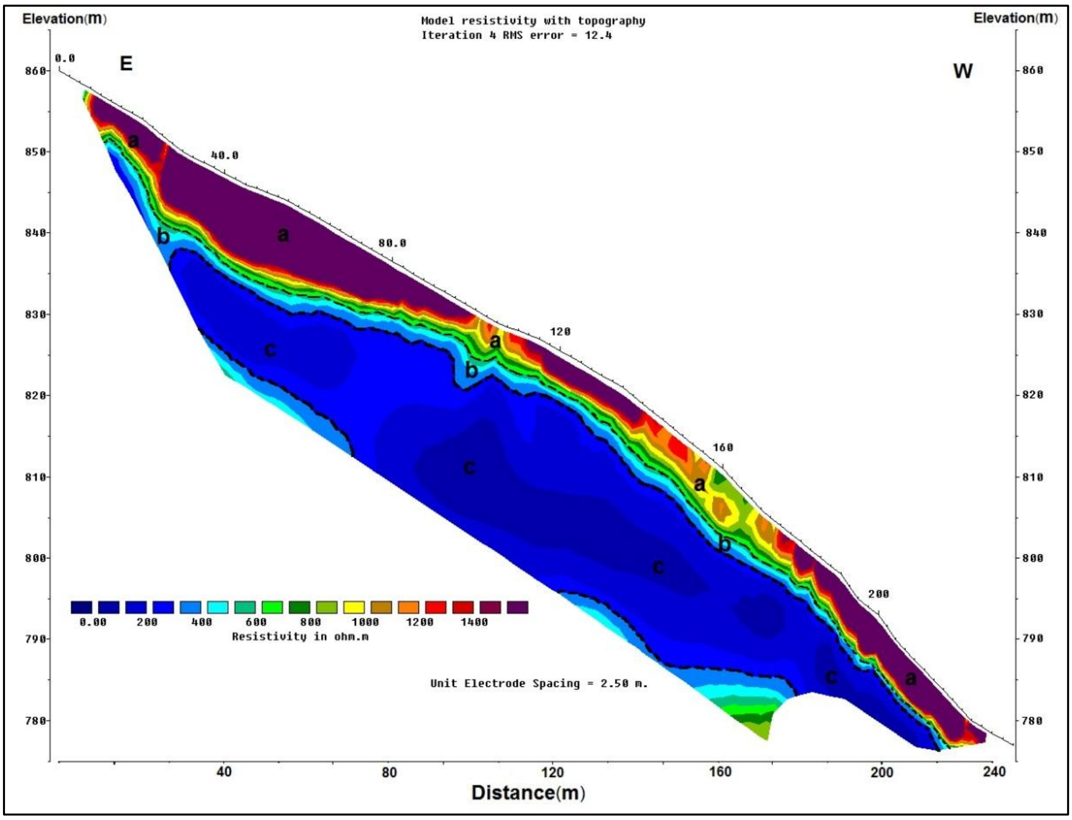

Figure 4.8. Result the resistivity with topography interpretation (BNP -2).

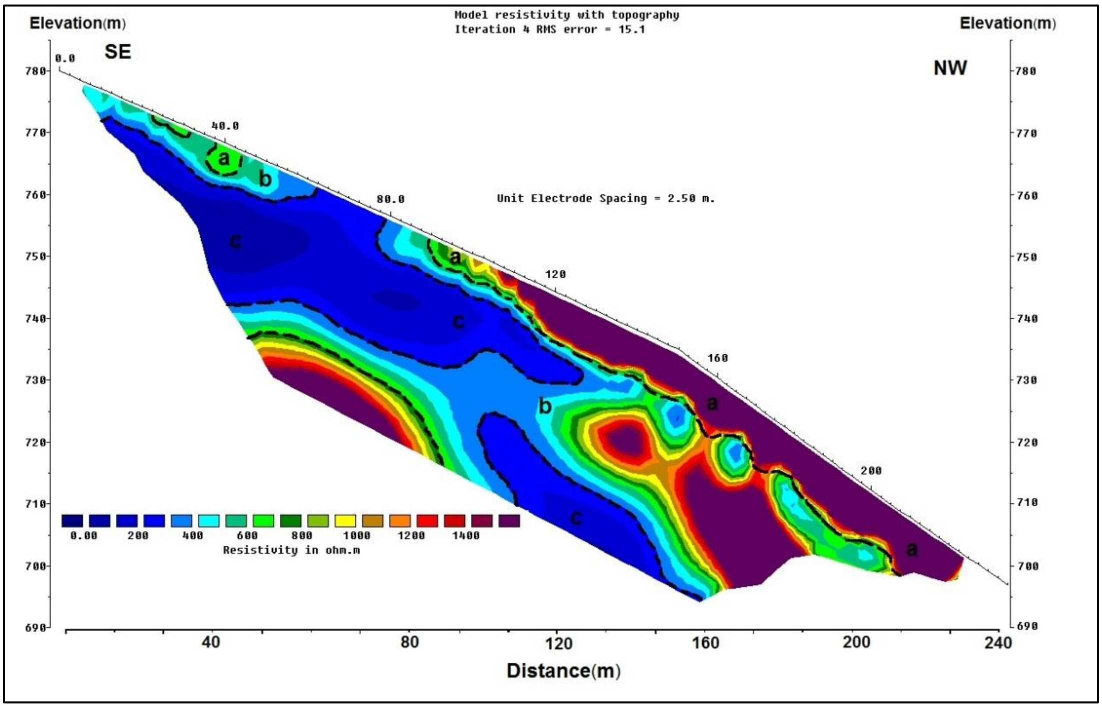

Figure 4.9. Result the resistivity with topography interpretation (BNP –3).

4.5 Test result physical properties

The physical properties of residual soil samples were analyzed in geotechnical engineering laboratory by the testing method of American Society for Testing and Materials (ASTM) including total unit weight, water content test (ASTM D 2216), specific gravity test (ASTM D 854), sieve analysis test (ASTM D 422), hydrometer test (ASTM D 422), Atterberg's limits test (ASTM 4318), and permeability test (ASTM D 2434-68). All test results will be explained as follows.

4.5.1 Grain size distribution

Grain size analysis helps to classify soils, especially coarse soils. It is possible to tell from grain size distribution analysis whether the soil consists of predominantly gravel, sand, silt, or clay, and to a limited extent, which of these size ranges is likely to control the soil engineering properties. Furthermore, collected the disturbed soil samples (11 samples included BNP 1 – BNP 11) for testing find the physical properties and soil permeability were tested at geotechnical laboratory with position of collected disturbed soil sample in study area (see figure 4.10). Disturbed soil samples are likely to be more representative in the area.

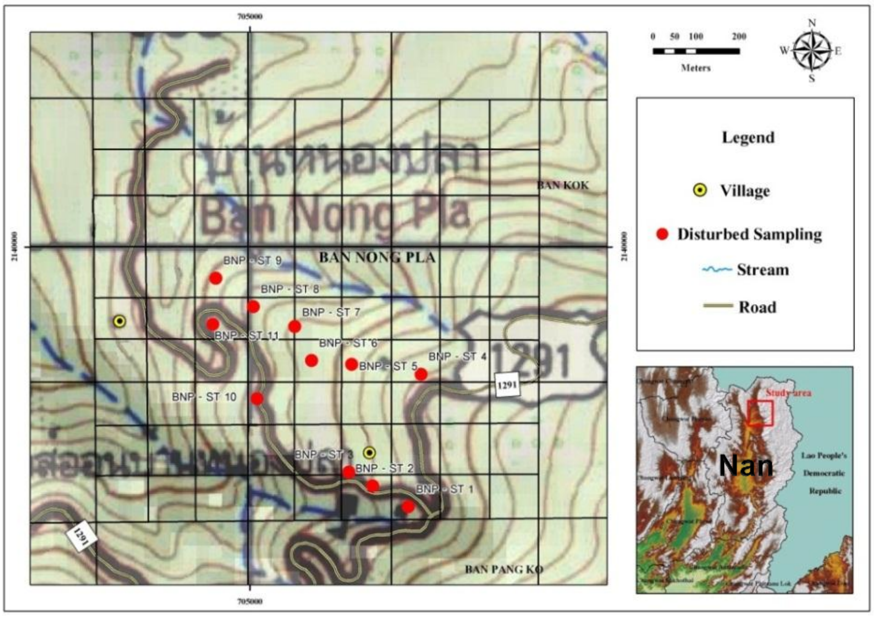

Figure 4.10. Position the collected disturbed soil sample.

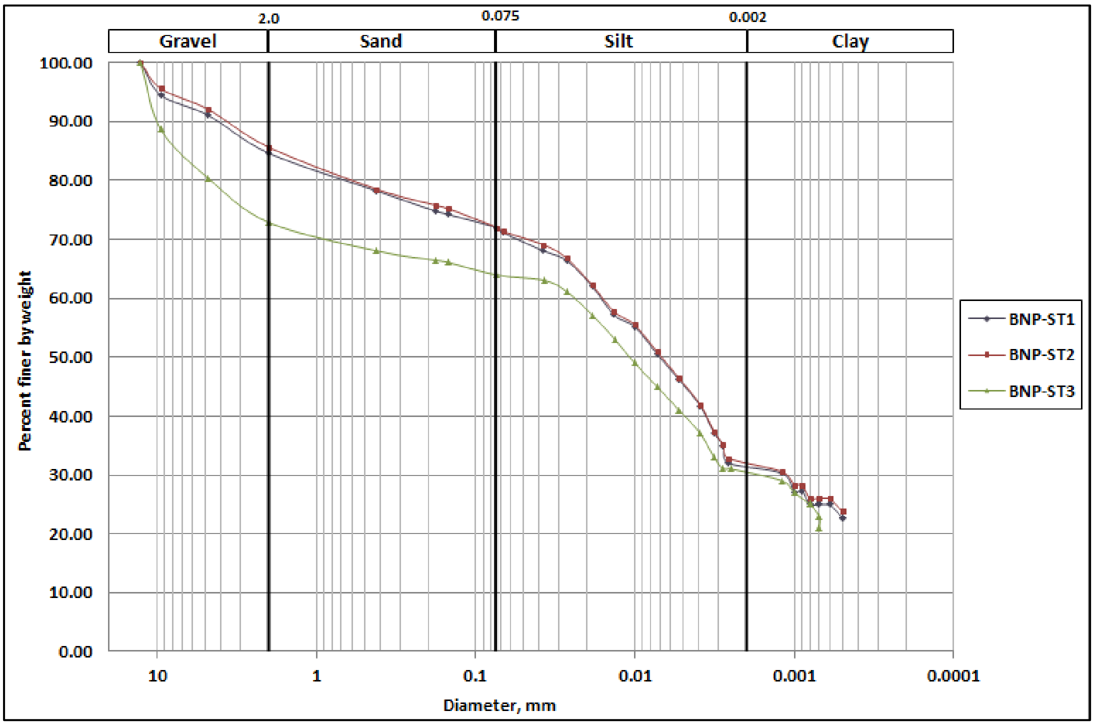

Figure 4.11. Test results grain size distribution BNP-ST 1, 2 ,3.

Grain size distribution curve is shown in figure 4.11 that can be summarized as follows.

1. Sample BNP-ST 1 has quantity of gravel as 15.34 percent, 12.64 percent sand, 41.74 percent silt and finally 30.28 percent clay.

2. Sample BNP- ST2 has quantity of gravel 14.34 percent, 13.62 percent sand, 41.46 percent silt and 30.58 percent clay.

3. Sample BNP-ST3 has quantity of gravel as 27.12 percent, 8.92 percent sand, silt 34.95 percent and 29.01 percent clay.

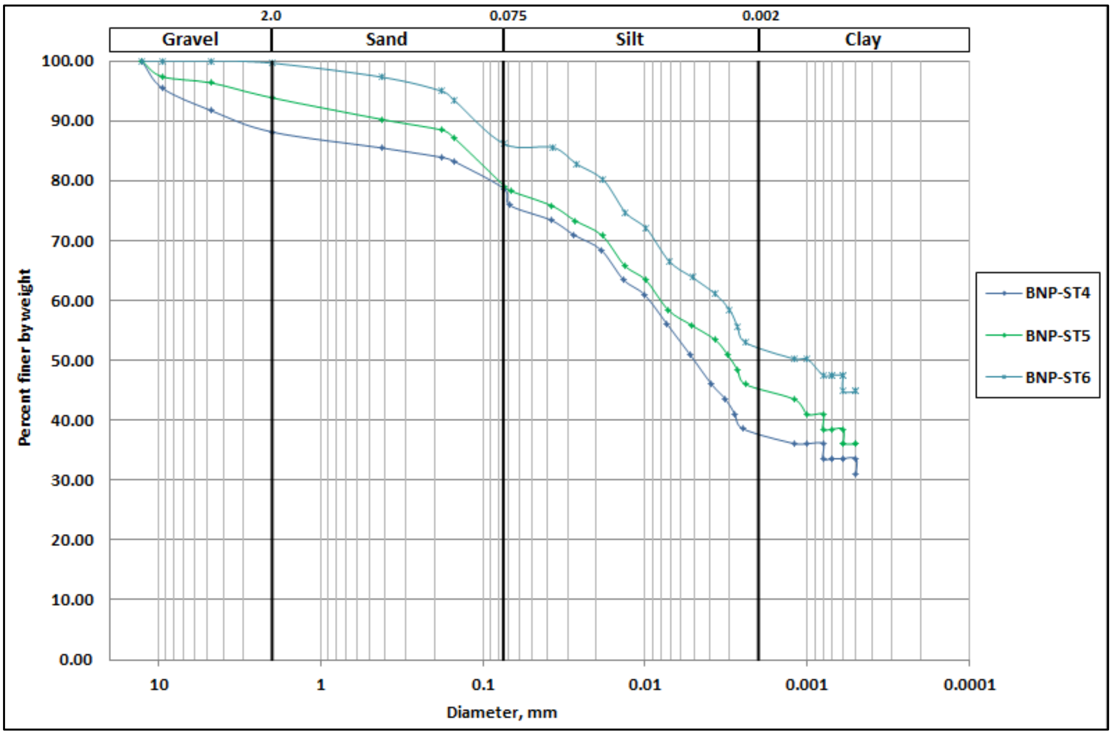

Figure 4.12. Test results grain size distribution BNP- ST 4,5,6.

Grain size distribution curve is shown in figure 4.12 that can be summarized as follows.

4. Sample BNP-ST 4 has quantity of gravel as 11.86 percent, 9.49 percent sand, 42.56 percent silt and finally 36.09 percent clay.

5. Sample BNP- ST5 has quantity of gravel 6.14 percent, 14.74 percent sand, 35.63 percent silt and 43.49 percent clay.

6. Sample BNP-ST6 has quantity of gravel as 0.37 percent, 13.49 percent sand, 35.89 percent silt and 50.25 percent clay.

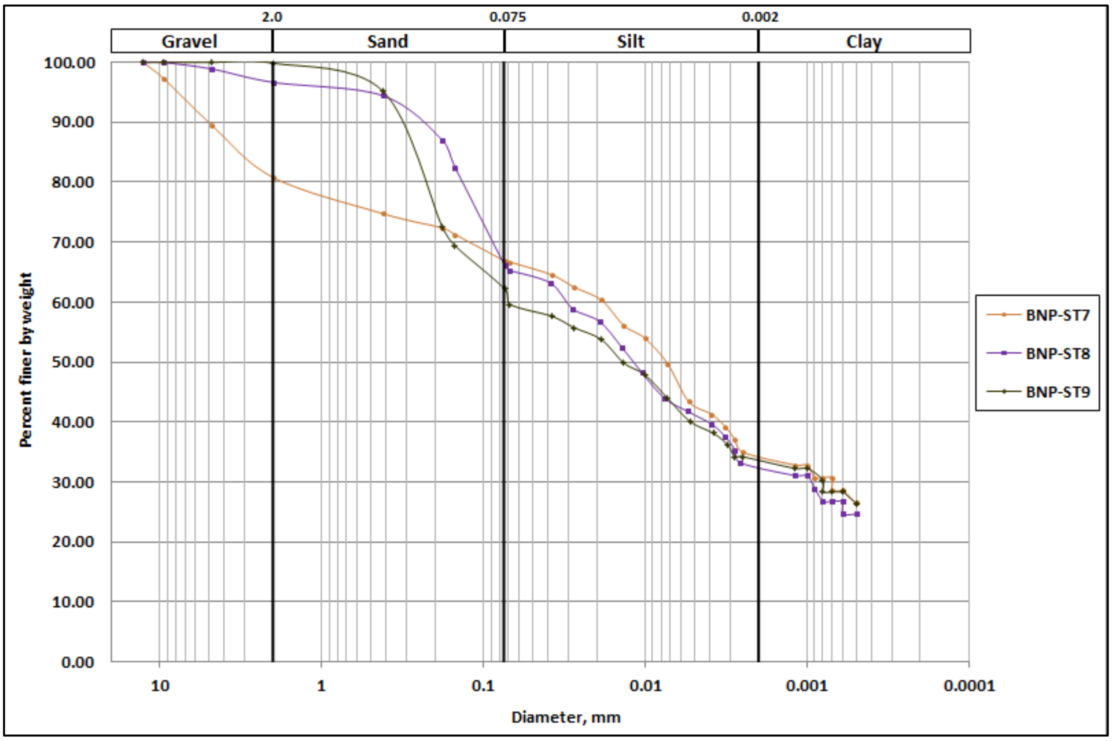

Figure 4.13. Test results grain size distribution BNP- ST 7,8,9.

Grain size distribution curve is shown in figure 4.13 that can be summarized as follows.

7. Sample BNP-ST 7 has quantity of gravel as 19.19 percent, 13.94 percent sand, 34.06 percent silt and finally 32.81 percent clay.

8. Sample BNP- ST8 has quantity of gravel 3.32 percent, 30.55 percent sand, 35.07 percent silt and 31.06 percent clay.

9. Sample BNP-ST9 has quantity of gravel as 0.12 percent, 37.71 percent sand, 34.95 percent silt and 32.25 percent clay.

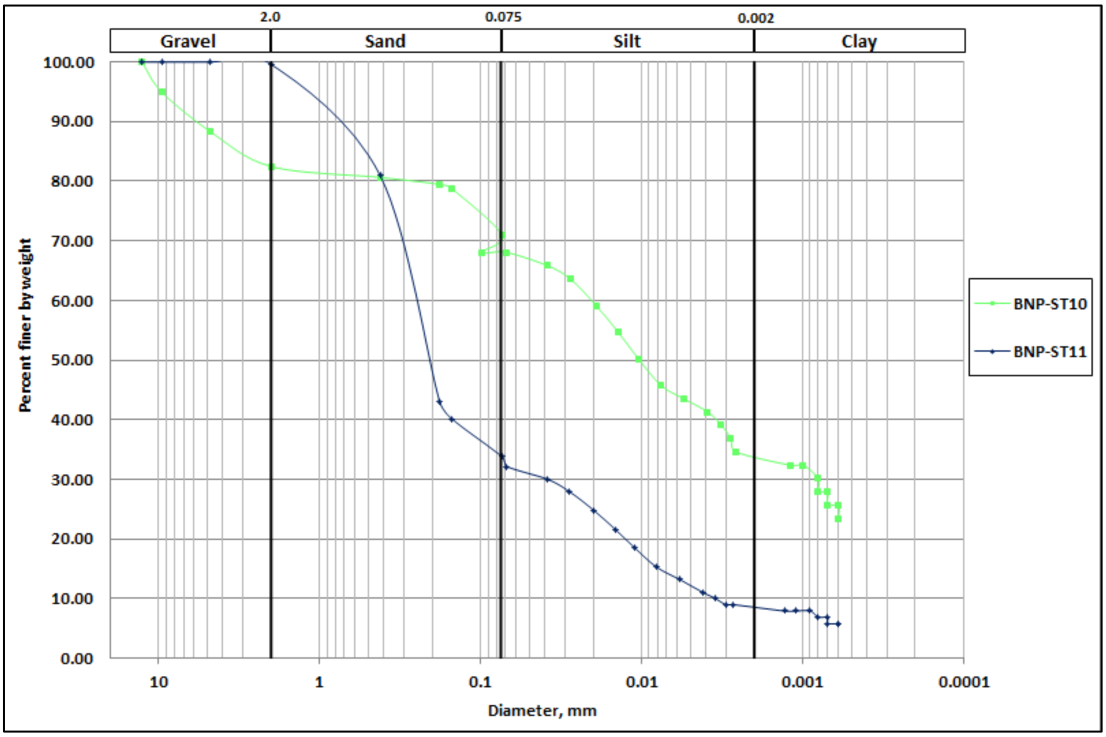

Figure 4.14. Test results grain size distribution BNP- ST 10,11.

Grain size distribution curve shown in figure 4.14 that can be summarized as follows.

10. Sample BNP-ST10 has quantity of gravel as 17.51 percentage, 11.48 percent sand , 38.64 percent silt and finally 32.37 percent clay.

11. Sample BNP-ST11 has quantity of gravel as 0.37 percent, 65.82 percent sand, 25.91 percent silt and 7.90 percent clay.

Table 4.2 Test results classification type of soil according to size and grain size distribution from disturbed soil sample amount 11 samples.

Parent rock	Test Pit	Type of soil				C_u	C_c
		Gravel > 2.0	Sand 2.0-0.075	Silt 0.075-0.002	Clay < 0.002		
Shale	BNP-ST1	15.34 %	12.64 %	41.74 %	30.28 %	-	-
Shale	BNP-ST2	14.34 %	13.62 %	41.46 %	30.58 %	-	-
Shale	BNP-ST3	27.12 %	8.92 %	34.95 %	29.01 %	-	-
Shale	BNP-ST4	11.86 %	9.49 %	42.56 %	36.09 %	-	-
Siltstone	BNP-ST5	6.14 %	14.74 %	35.63 %	43.49 %	-	-
Siltstone	BNP-ST6	0.37 %	13.49 %	35.89 %	50.25 %	-	-
Shale	BNP-ST7	19.19 %	13.94 %	34.06 %	32.81 %	-	-
Siltstone	BNP-ST8	3.32 %	30.55 %	35.07 %	31.06 %	-	-
Siltstone	BNP-ST9	0.12 %	37.71 %	29.92 %	32.25 %	-	-
Shale	BNP-ST10	17.51%	11.48 %	38.64 %	32.37 %	-	-
Sandstone	BNP-ST11	0.37 %	65.82 %	25.91 %	7.9 %	51.42	0.0083

C_u is Uniformity Coefficient , C_c is Coefficient of gradation

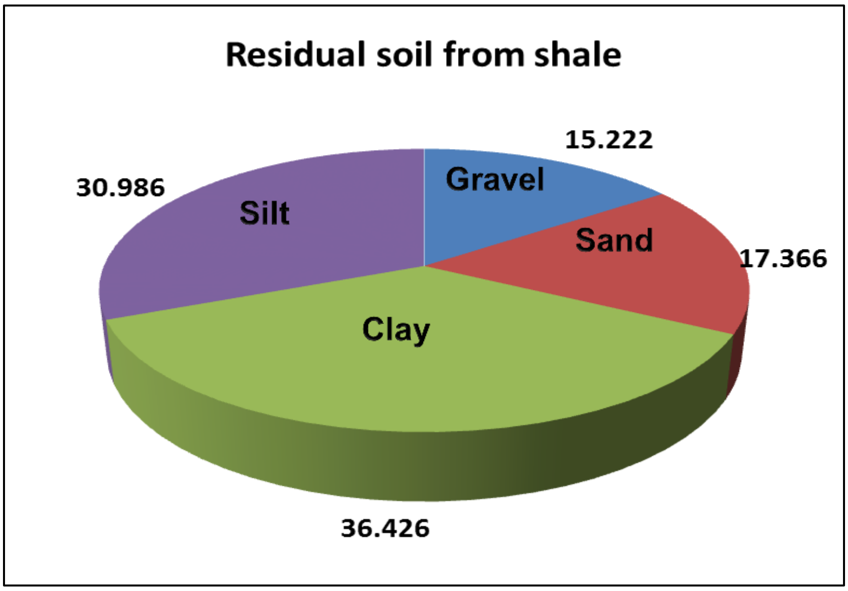

Figure 4.15. Test results grain size distribution of residual soil from shale.

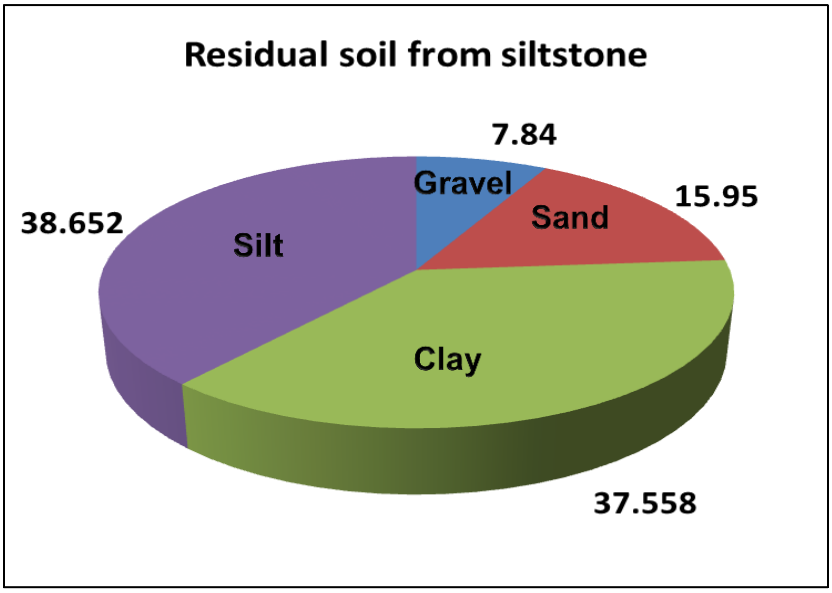

Figure 4.16. Test results grain size distribution of residual soil from siltstone.

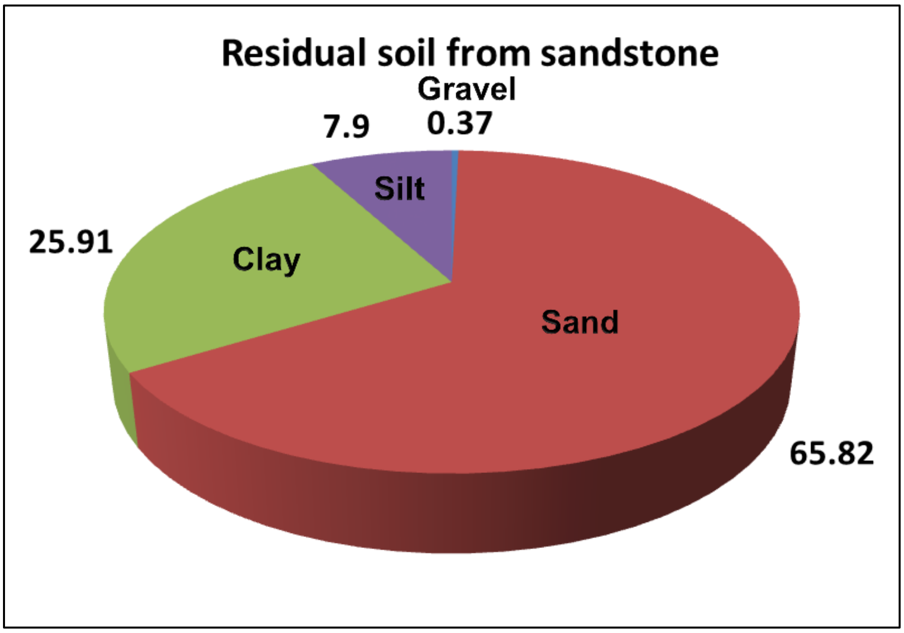

Figure 4.17. Test results grain size distribution of residual soil from sandstone.

From figures 4.15 to 4.17, it can be concluded that residual soil from shale has quantity of gravel 15.22 percent, 17.36 percent sand, 30.98 percent silt and 36.42 percent clay. residual soil from siltstone has quantity of gravel 7.84 percent, 15.95 percent sand, 38.65 percent silt and 37.55 percent clay. And residual soil from sandstone has quantity of gravel 0.37 percent, 65.82 percent sand, 7.9 percent silt and 25.91 percent clay

Testing results of grain size distribution from 11 samples can be concluded that residual soil from weathered parent rocks such as shale, siltstone and sandstone occurred in this study area has high percentage of fine grained texture and can be qualified to poor drainage, thus making soil adsorb water during raining. Therefore, shear strength likely decreases thus, providing high opportunity to slide.

4.5.2 Water content & Total unit weight

Testing the water content (ASTM D 2216) and total unit weight based on American Society for Testing and Materials (ASTM) is shown in the Table 4.3.

Table 4.3. Results of testing water content and total unit weight of disturbed soil sample according to weathered parent rock.

Parent rock	Test Pit	Water content %	Total unit weight (t /m^3)
Shale	BNP-ST1	20.31	1.805
Shale	BNP-ST2	26.91	1.590
Shale	BNP-ST3	30.52	1.760
Shale	BNP-ST4	31.74	1.870
Siltstone	BNP-ST5	27.89	1.704
Siltstone	BNP-ST6	22.66	1.750
Shale	*BNP-ST7	25.76	-
Siltstone	BNP-ST8	20.92	1.777
Siltstone	BNP-ST9	17.52	1.677
Shale	BNP-ST10	20.56	1.532
Sandstone	BNP-ST11	12.06	1.462

Remark : Sample *BNP – ST 7 Can not collected disturbed sample for testing total unit weight

4.5.3 Specific gravity , void ratio and porosity

Testing a specific gravity (ASTM D 854) and porosity of soil is based on American Society for Testing and Materials (ASTM) is shown in the following table.

Table 4.4 Results of testing specific gravity, void ratio, porosity of disturbed soil sample according to weathered parent rock.

Parent rock	Test Pit	Specific gravity	Void ratio	Porosity %
Shale	BNP-ST1	2.67	0.804	44.56
Shale	BNP-ST2	2.67	1.076	51.86
Shale	BNP-ST3	2.69	0.880	46.80
Shale	BNP-ST4	2.69	0.914	47.74
Siltstone	BNP-ST5	2.69	1.013	50.32
Siltstone	BNP-ST6	2.70	0.950	48.70
Shale	*BNP-ST7	2.66	-	-
Siltstone	BNP-ST8	2.66	0.796	44.31
Siltstone	BNP-ST9	2.67	0.850	45.79
Shale	BNP-ST10	2.67	0.898	46.56
Sandstone	BNP-ST11	2.70	1.066	52.10

Remark : Sample *BNP – ST 7 Can not collected disturbed sample for testing void ratio and porosity

4.5.4 Atterberg's limits

The engineering behavior of fine-grained soils depended on factors other than particle size distribution and density. It is influenced primarily by their mineral and structural composition and the amount of water they contain, which is referred to as water content. The liquid and plastic limits tests characterize the effects of water content on fined-grained soils on their consistency (liquid, plastic, semi-solid and solid) in disturbed condition and help to classify fine-grained soils and to assess their mineral composition.

A fine-grained soil can exist in any of several states; which state depends on the amount of water in the soil system. When water is added to dry soil, each particle is covered with a film of absorbed and double layer water. If the addition of water is continued, the thickness of the water film on a particle increases. Increasing the thickness of the water films permits the particles to slide past one another more easily.

The plasticity value of soil can test by "Atterberg's limit " which find the liquid limits and plastic limits for adopt find toughness properties of soil and using to classification category of soil in fine grained texture such as silty soil to clayey soil. By plot the Liquid limits (LL) value and Plastic index (PI) is shown in figures 4.18. and 4.19. Test result shown in the following table.

Table 4.5 Results of testing consistency of disturbed soil sample according to weathered parent rock .

Parent rock	Test Pit	Water content %	Atterberg's limits		
			LL	PL	PI
Shale	BNP-ST1	20.31	46.56	29.86	16.70
Shale	BNP-ST2	26.91	45.33	28.92	16.41
Shale	BNP-ST3	30.52	47.35	30.55	16.80
Shale	BNP-ST4	31.74	47.35	33.94	13.41
Siltstone	BNP-ST5	27.89	45.57	31.69	13.88
Siltstone	BNP-ST6	22.66	46.19	28.33	17.86
Shale	BNP-ST7	25.76	45.37	27.05	18.32
Siltstone	BNP-ST8	20.92	38.33	26.26	12.07
Siltstone	BNP-ST9	17.52	39.88	25.97	13.91
Shale	BNP-ST10	26.56	47.35	30.96	16.39
Sandstone	*BNP-ST11	22.36	-	-	-

Remark : *BNP - ST 11 is Non plastic

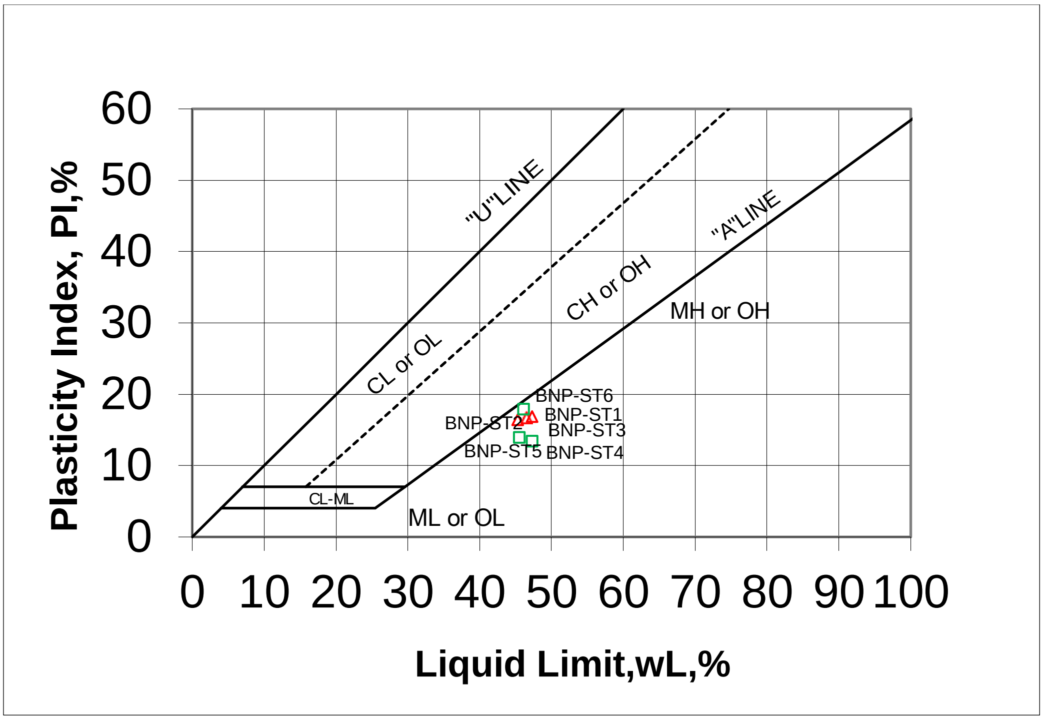

Figure 4.18. Test results consistency of sample BNP- ST 1,2,3,4,5,6.

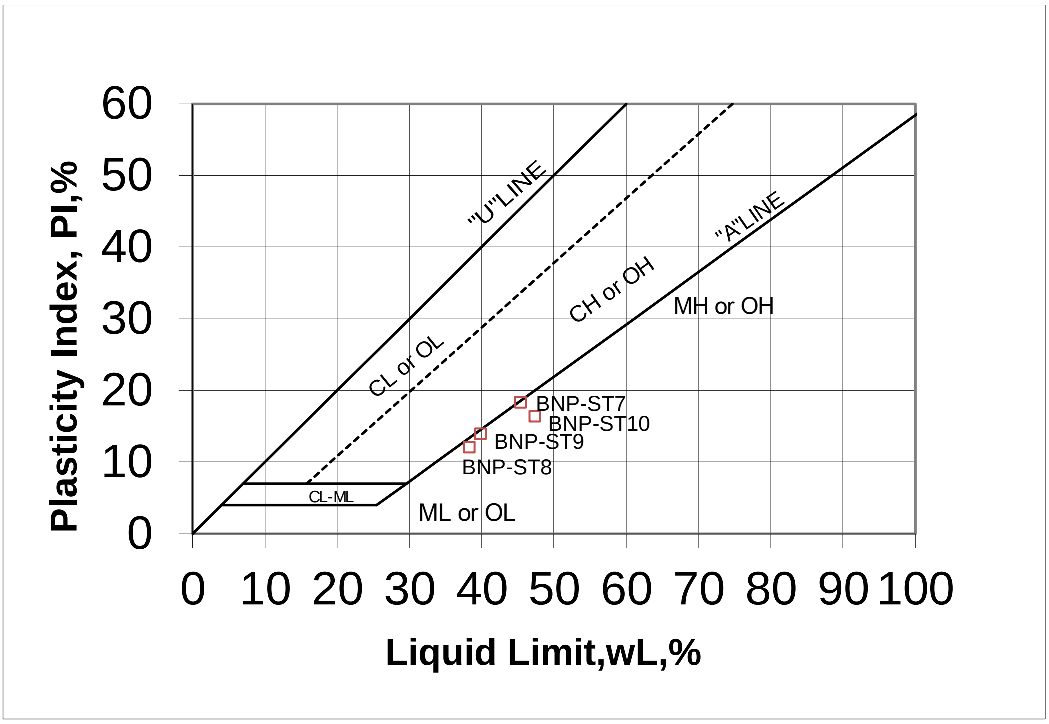

Figure 4.19. Test results consistency of sample BNP- ST 7,8,9,10 (continue).

Testing results of Atterberg's limit from 11 samples from parent rock can be concluded that most of residual soils in the study area owns the low plasticity (LL< 47 %) which is meant that the toughness properties of soil will be changed whenever the moisture content increases, event just a little increase (Hazelton and Murphy, 2007). This soil type is risky to cause the landslide.

4.5.5 Permeability of soil

Testing permeability of soil (ASTM D2434) was based on American Society for Testing and Materials (ASTM). Test result is shown in the following table 4.6.

Table 4.6 Results of testing permeability of recompacted soil material according to weathered parent rock

Parent rock	Test Pit	Permeability (cm/sec)
Shale	BNP-ST1	3.77×10^{-7}
Shale	BNP-ST2	1.01×10^{-7}
Shale	BNP-ST3	6.81×10^{-7}
Shale	BNP-ST4	1.52×10^{-7}
Siltstone	BNP-ST5	3.46×10^{-7}
Siltstone	BNP-ST6	1.37×10^{-7}
Shale	*BNP-ST7	-
Siltstone	BNP-ST8	3.32×10^{-7}
Siltstone	BNP-ST9	3.68×10^{-7}
Shale	BNP-ST10	6.29×10^{-7}
Sandstone	BNP-ST11	9.36×10^{-6}

Remark : Sample *BNP – ST 7 Can not collected disturbed sample for permeability test

Permeability refers to the propensity of material to allow fluid to move through its pores or interstices. In the context of soil, permeability generally relates to the propensity of soil to allow water to move through its void spaces.

Testing result the values coefficient permeability each in recompacted soils sample have values coefficient permeability close to each other and in the ranges values coefficient permeability of silty sand soil and clayey soil between 10^{-6} cm/sec to 10^{-7} cm/sec. Therefore, whenever water from rain fall to the soil surface with a lower rate than permeability of soil. Water from rain will seep into soil surface and when water from rain fall to the soil surface with a higher rate than permeability of soil water level in soil layer will increase, and then causing the decrease in stability of soil slope to become risk for landslide.

4.6 Classification of soil

The principal objective of any soil classification system is to predict the engineering properties and behavior of a soil based on a few simple laboratory or field tests. Laboratory or field test results are then used to identify the soil and put it into a group that has soils with similar engineering characteristics.

4.6.1 Unified soil classification system (USCS)

The Unified Soil Classification System (USCS) can be classified into the major soil categories by letter symbols, such as S for sand, G for gravel, M for silt, C for clay. According to the Unified Soil Classification System (USCS).

Table 4.7. Results of soil classification using unified soil classification system according to weathered parent rock.

Parent rock	Test Pit	USCS
Shale	BNP-ST1	ML- CL
Shale	BNP-ST2	ML- OL
Shale	BNP-ST3	ML- CL
Shale	BNP-ST4	ML- CL
Siltstone	BNP-ST5	CL- ML
Siltstone	BNP-ST6	CL- ML
Shale	BNP-ST7	ML- OL
Siltstone	BNP-ST8	ML- CL
Siltstone	BNP-ST9	SC
Shale	BNP-ST10	ML- CL
Sandstone	BNP-ST11	SM

From table 4.7, it can be summarized as follows:

1. Residual soil from shale including the sample BNP-ST1, BNP-ST2, BNP-ST3, BNP-ST4 BNP-ST7, BNP-ST10 can be classified into ML-CL, ML-OL, ML-CL, ML-CL and ML-CL, respectively. Because mostly disturbed soils sample are silty clay and clayey soil, silty clays and organic silt clays soil.

2. Residual soil from siltstone including the sample BNP-ST5, BNP-ST6, BNP-ST8, BNP-ST9 can be classified into ML-CL, CL-ML, CL-ML, ML-CL, SC, respectively. Because mostly disturbed soils sample are silty clay and clayey soil, clayey fine soil and silty clay, clayey sands and poorly graded sand-clay mixtures.

3. Residual soil from sandstone including the sample BNP-ST11 can be classified into SM. Because disturbed soil sample is silty sand soil and poorly graded and that is not plasticity (Non plastic).

Therefore, it can be concluded that mostly residual soils from weathered parent rock in study area can be classified as ML-CL, CL-ML and SM. Silty sand soil to clayey fine soil occurred in this study area includes high percentage of fine grained texture that can be qualified to poor drainage, thus making soil adsorb water during raining. Therefore, shear strength likely decreases thus, providing high opportunity to slide.

4.7 Test result engineering properties

This research will focus on analyzing engineering properties of residual soil from weathered parent rock. Collected undisturbed soil samples were tested at Geotechnical laboratory to find the properties of soil shear strength in natural condition and properties of the soil shear strength decreases when the moisture content increases.

Undisturbed soil sample is collected at the interface level between bed rock and residual soil for testing find the shear strength parameter (cohesion and friction angle) including consolidation drained test (8 test pits included TP 1 - TP 8) and consolidation undrained test (1 test pits included TP 9) with location of collected undisturbed soil sample in study area (see figure 4.20). The testing method multi-stage direct shear test can be performed find shear strength parameter by use one example only.

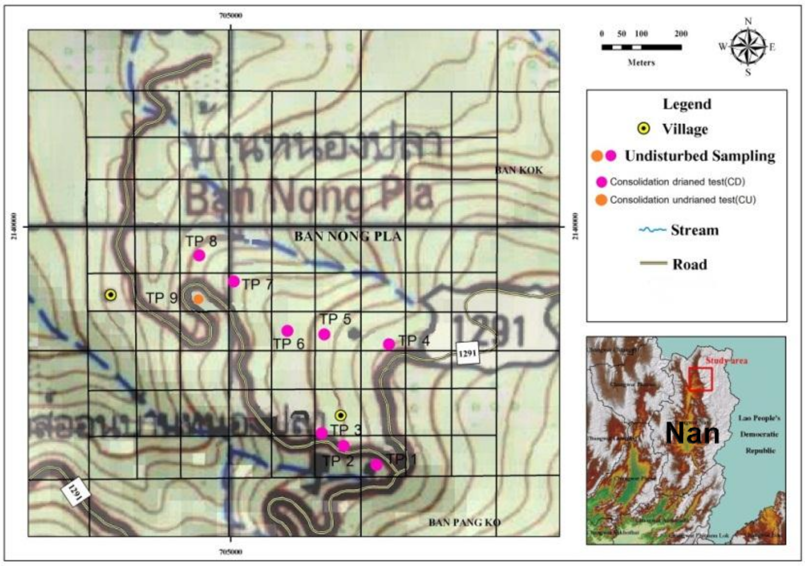

Figure 4.20. location of collected Undisturbed soil sample in study area.

Multi - stage direct shear test is applied to test shearing sample until nearly to the failure point in each normal load (at least three or four normal load). This testing method is suitable for sample with high variability and in case less sample (Mairaing, 2008). This testing method will give the value shear strength parameter reliable more than the conventional test. The typical results from multistage direct shear test with moisture content variation is shown in figure 4.21, 4.22 (CD test) and figure 4.23, 4.24 (CU test).

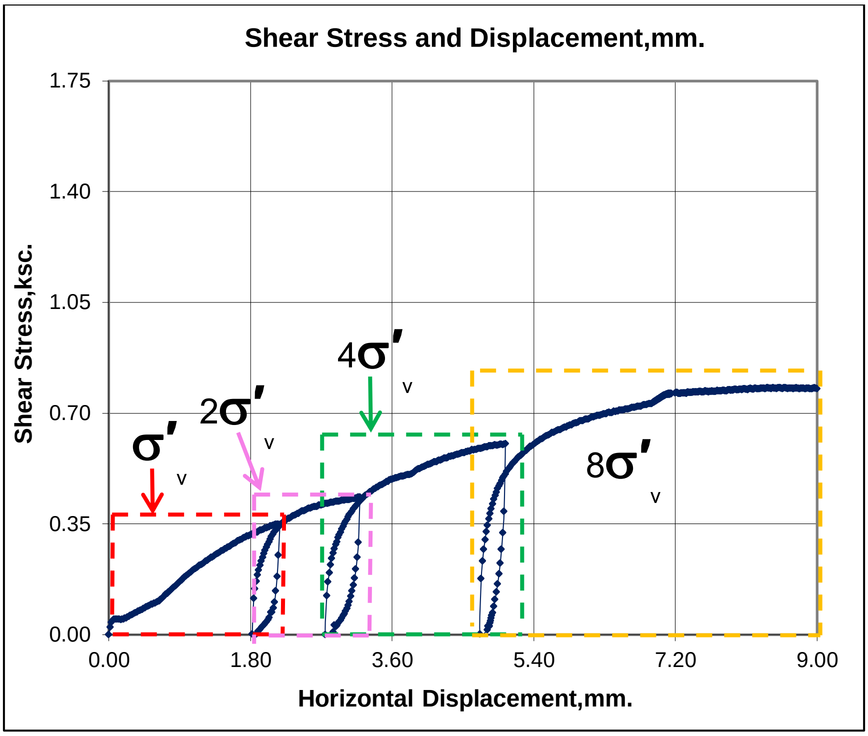

Figure 4.21. Result of testing relationship between shear stress and horizontal displacement.

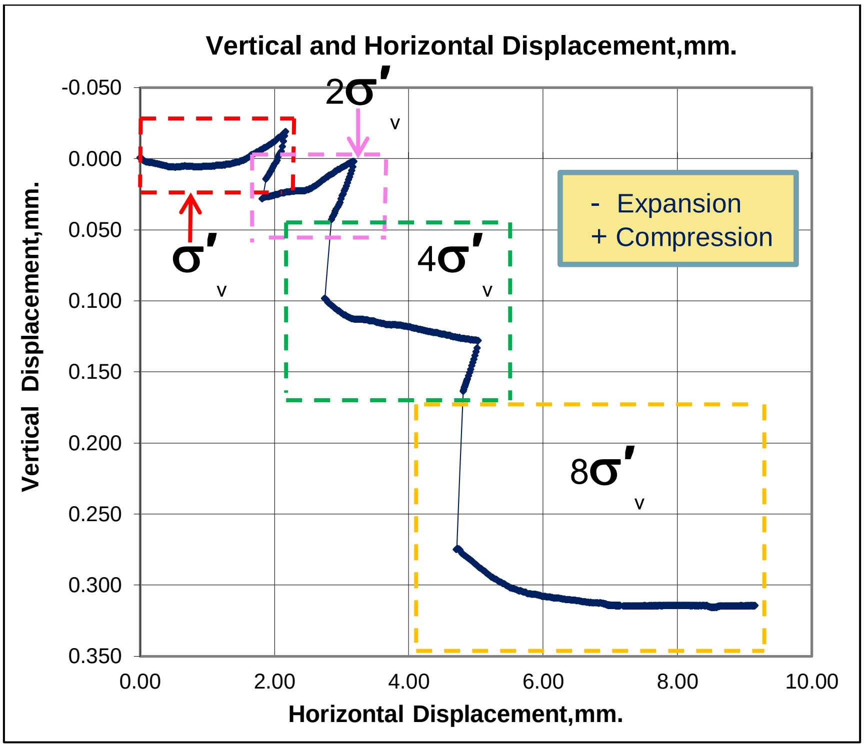

Figure 4.22. Result of testing relationship between vertical displacement and horizontal displacement.

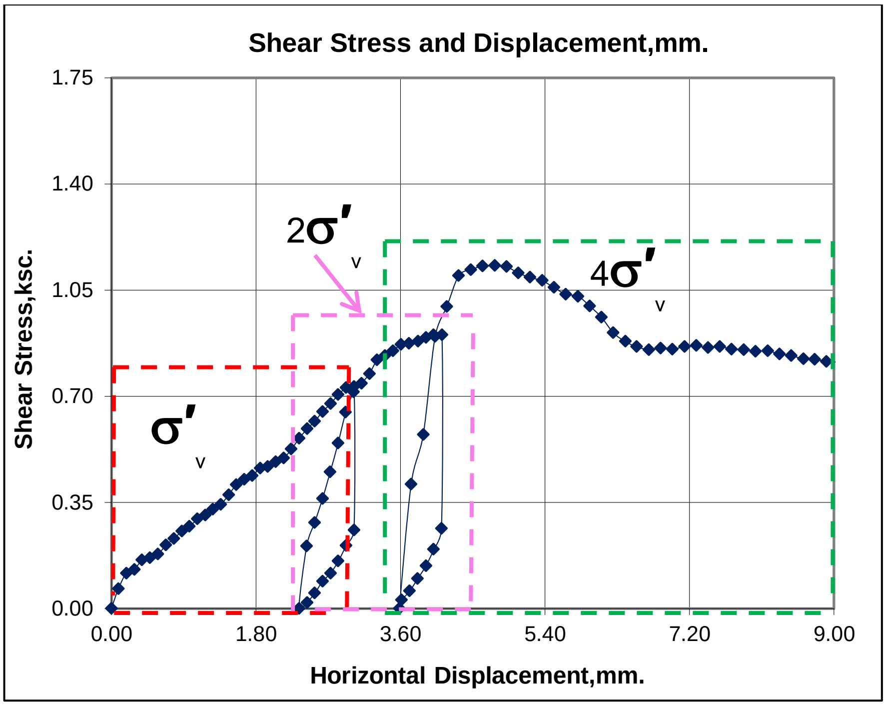

Figure 4.23. Result of testing relationship between shear stress and horizontal displacement.

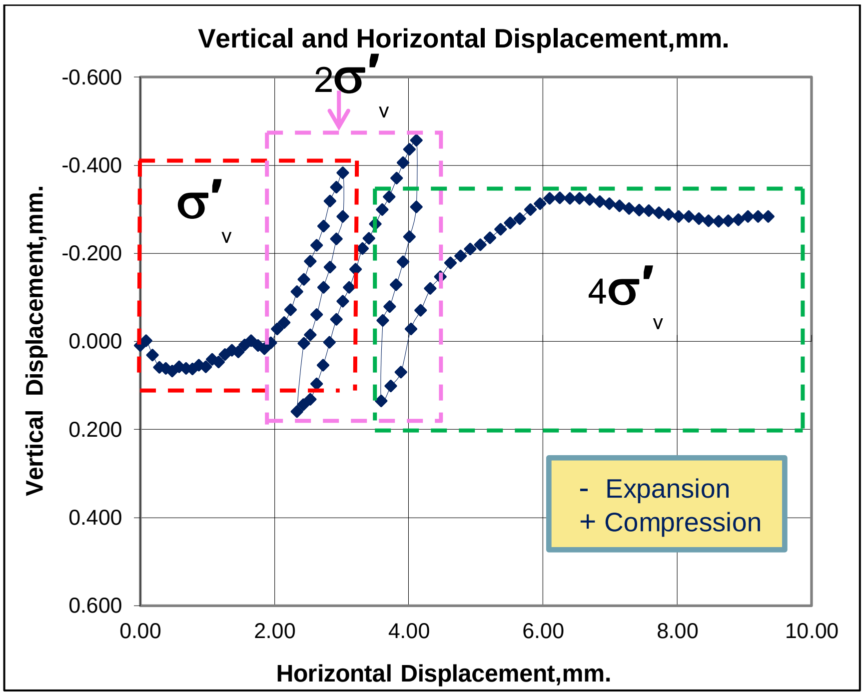

Figure 4.24. Result of testing relationship between vertical displacement and horizontal displacement.

4.7.1 Initial water content condition

Shear strength of residual soils from this study area was done by " Multi-stage direct shear test". The testing shear strength of residual soil at natural moisture content condition is to define a range of moisture content that has changed. Testing result multi-stage direct shear test (CD test and CU test) at natural moisture content is shown from Mohr diagram in figure 4.25, 4.26 and 4.27. and the strength parameter (cohesion and friction angle) is summarized in tables 4.8 and 4.9.

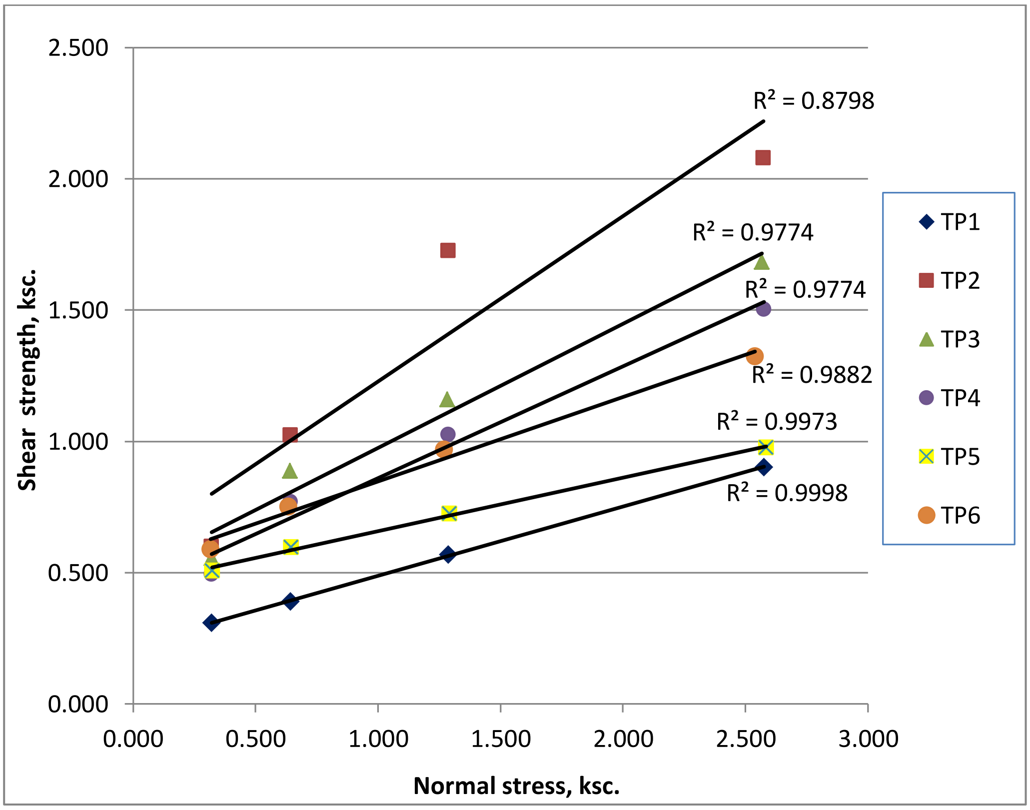

Figure 4.25. Soil shear strength values of undisturbed soil sample at natural moisture content amount 6 Test pit (CD test).

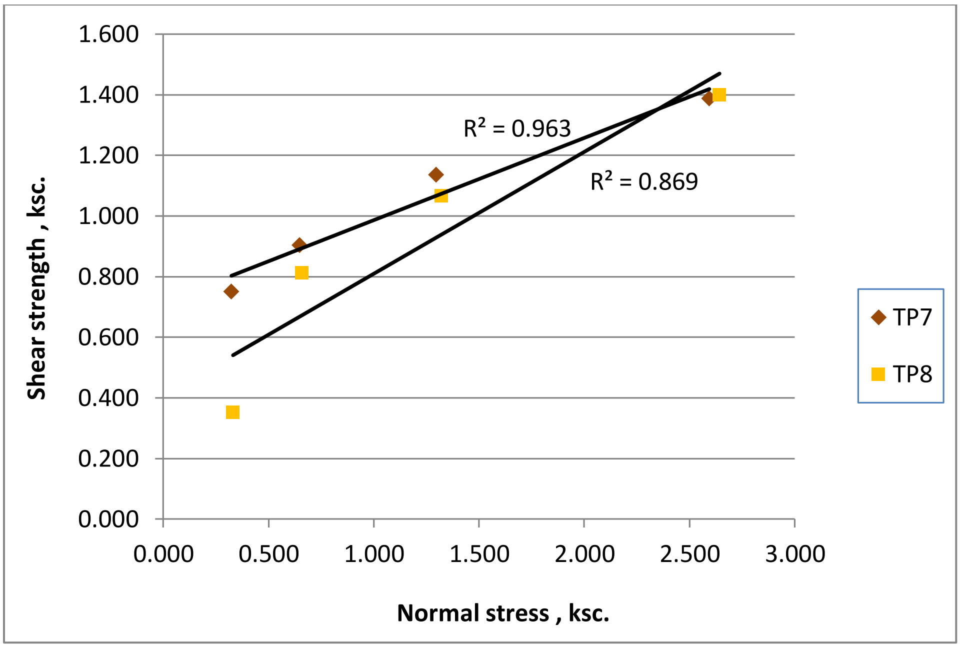

Figure 4.26. Soil shear strength values of undisturbed soil sample at natural moisture content from 2 Test pit (CD test) continue.

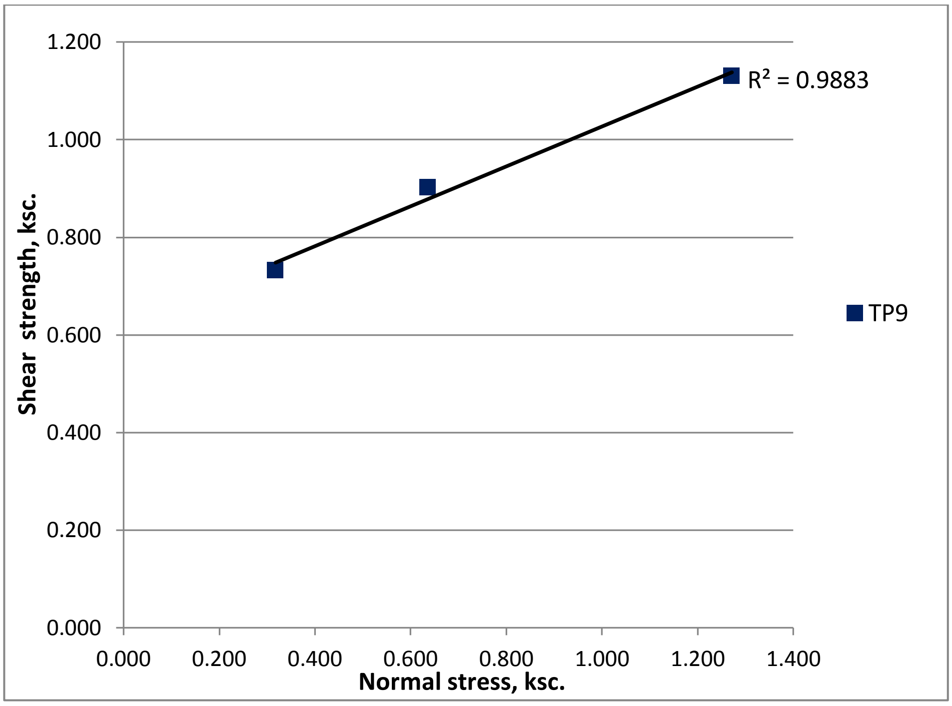

Figure 4.27. Soil shear strength values of undisturbed soil sample at natural moisture content form 1 Test pit (CU test).

Table 4.8. Results of testing shear strength of soil at natural moisture content conditions using the test pattern consolidation drained test.

Test pit	WC %	Depth(m)	Degree of weathering	c′ (ksc.)	ϕ′ (Degree)
TP 1	22.02	2.0	Grade V	0.223	14.77
TP 2	23.75	2.5	Grade V	0.315	20.47
TP 3	22.85	2.5	Grade V	0.501	25.30
TP 4	33.04	0.8	Grade VI	0.433	23.08
TP 5	27.57	0.6	Grade V	0.455	11.51
TP 6	26.87	1.0	Grade VI	0.525	17.82
TP 7	19.98	0.8	Grade VI	0.714	15.17
TP 8	15.81	1.2	Grade VI	0.408	21.90

From table 4.8, it can be concluded that the consolidation drained direct shear tests in the natural water content condition are clearly shown that cohesion value has tendency of increasing when the degree of saturation has low value. The cohesion of soil value is as 0.223 to 0.714 ksc and friction angle is as 11.51 to 25.30 degree.

Table 4.9. Results of testing shear strength of soil at natural moisture content conditions using the test pattern consolidation undrained test.

Test pit	WC %	Depth(m)	Degree of weathering	c (ksc.)	ϕ(Degree)
TP 9	11.97	3.0	Grade V	0.617	22.29

From table 4.9, it can be concluded that the strength parameter in the natural water content condition are clearly shown that cohesion value has tendency of increasing when the degree of saturation has low value .The Cohesion of soil value as 0.617 ksc and Friction angle as 22.29 degree.

4.7.2 Varies degree of saturation

The results of testing the effective shear strength of residual soil in vary degree of saturation, S_r in 3 levels: 60%, 80% and 100% or KU-MDS test (Soralump and Thowiwat , 2009) is applied for finding the properties of the soil shear strength decreases when the moisture content increases. After the testing multi-stage direct shear at natural moisture content of all test pit after that preparing the sample at moisture content levels of degree of saturation and testing multi-stage direct shear of sample in each test pit. Testing result is shown in table 4.10 for consolidation drained test and table 4.11 for consolidation undrained test.

Table 4.10. Results of testing shear strength of soil at degree of saturation using the test pattern consolidation drained test (KU – MDS test)

Degree of saturation, %	40 - 60		60 - 75		75 - 90		90 - 100	
	c'	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'
TP 1	0.31	20.47	0.28	12.33	0.22	14.77	0.19	21.58
TP 4	1.19	20.84	0.45	11.51	0.43	23.08	0.09	35.78
TP 8	0.40	21.9	0.71	15.17	0.32	13.08	0.14	19.79

Table 4.10, it can be concluded that the consolidation drained direct shear tests at varies degree of saturation (KU-MDS test) are clearly shown that cohesion value has tendency of decreasing when the degree of saturation has high value. The Cohesion of soil value as 0.09 ksc to 1.19 ksc and Friction angle as 11.51 degree to 35.78 degree.

Table 4.11 Results of testing shear strength of soil at degree of saturation using the test pattern consolidation undrained test (KU – MDS test).

Degree of saturation, %	30 - 50		60 - 75		75 - 90		90 - 100	
	c	ϕ	c	ϕ	c	ϕ	c	ϕ
TP 9	0.61	22.28	0.53	29.8	0.43	19.45	0.18	25.34

Table 4.11, it can be concluded that he consolidation undrained direct shear tests at varies degree of saturation (KU-MDS test) are clearly shown that cohesion value has tendency of decreasing when the degree of saturation has high value. The Cohesion of soil value as 0.18 ksc to 0.61 ksc and Friction angle as 19.45 degree to 29.8 degree.

Figure 4.28, 4.29 (CD test) and Figure 4.30, 4.31 (CU test) shown that relationship between cohesion and friction angle with degree of saturation in each Test Pit found that when the degree of saturation increasing the total cohesion value will decreases but the effective friction angle value doesn't change when the degree of saturation increases or the degree of saturation increasing no impact to effective friction angle. And consider the effective friction angle value that have most valuable that is Test Pit : TP 4 and Test Pit : TP 1 and TP 8 have the value lower, respectively. And when considering the cohesion value with percentage of fine grained soil in each Test Pit will found that Test Pit that has percentage of fine grained soil high will has the cohesion value higher. Which Test Pit : TP 4 has the most value cohesion and followed by TP 5 , TP 6 ,TP 1 , TP 2 ,TP 3,TP 7,TP 8 and TP 9 which have percentage of fine grained soil less, respectively. considering from table 4.1. With the cohesion value in each Test Pit will has the values less it began is zero at the degree of saturation value about 83 – 95 % because water entered in the void increase so flow connected together thus making surface tension be destroyed resulted the cohesion value decreases.

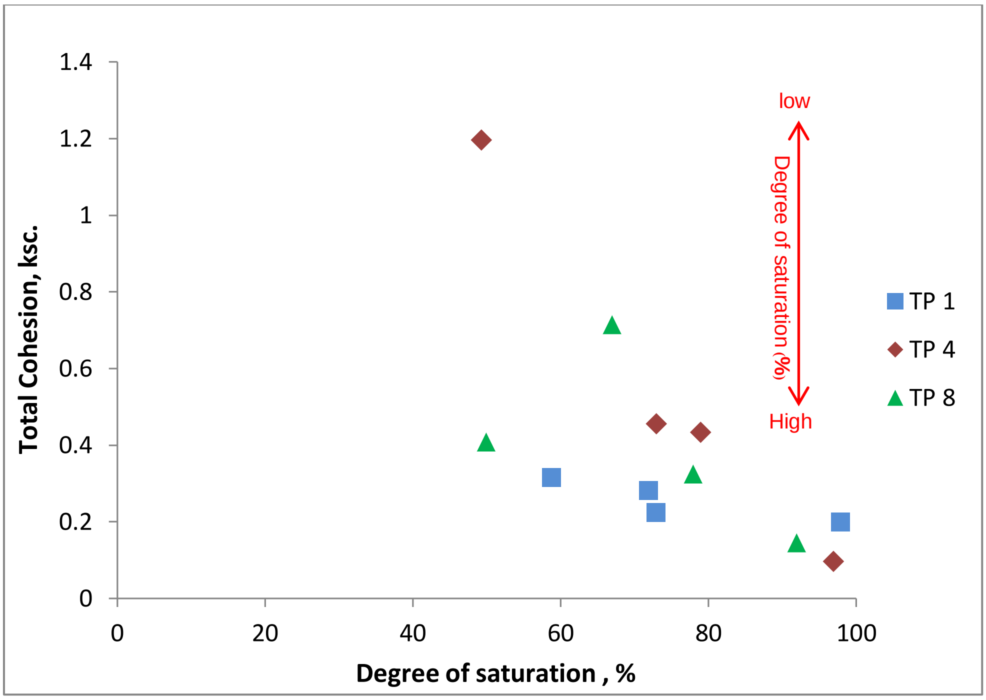

Figure 4.28. Test results relationship between cohesion and degree of saturation in each Test pit (CD test).

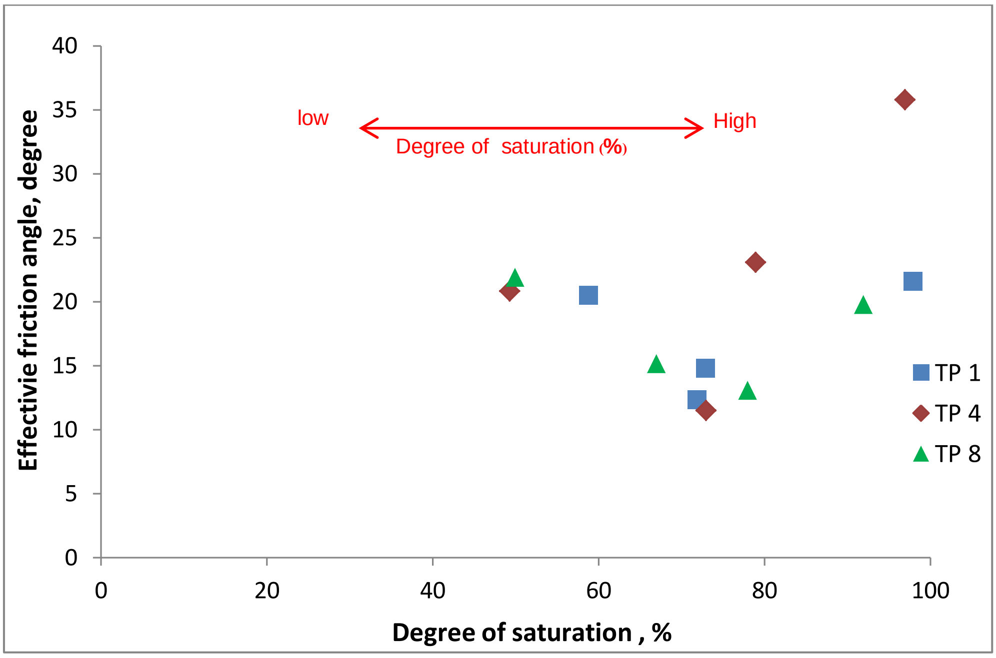

Figure 4.29. Test results relationship between effective friction angle and degree of saturation in each Test pit (CD test).

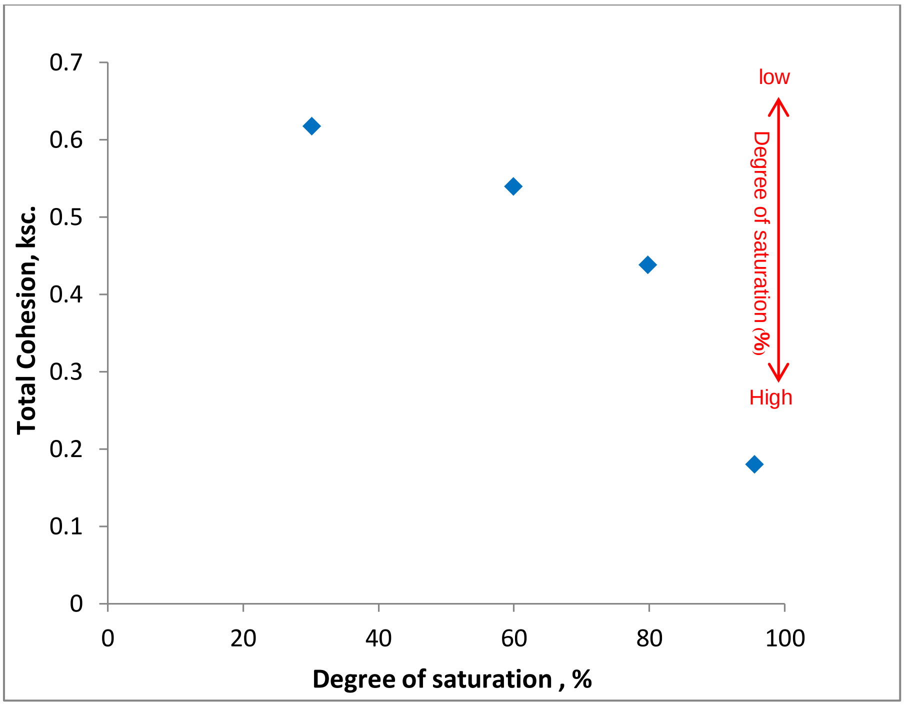

Figure 4.30. Test results relationship between cohesion and degree of saturation in Test pit : TP 9 (CU test).

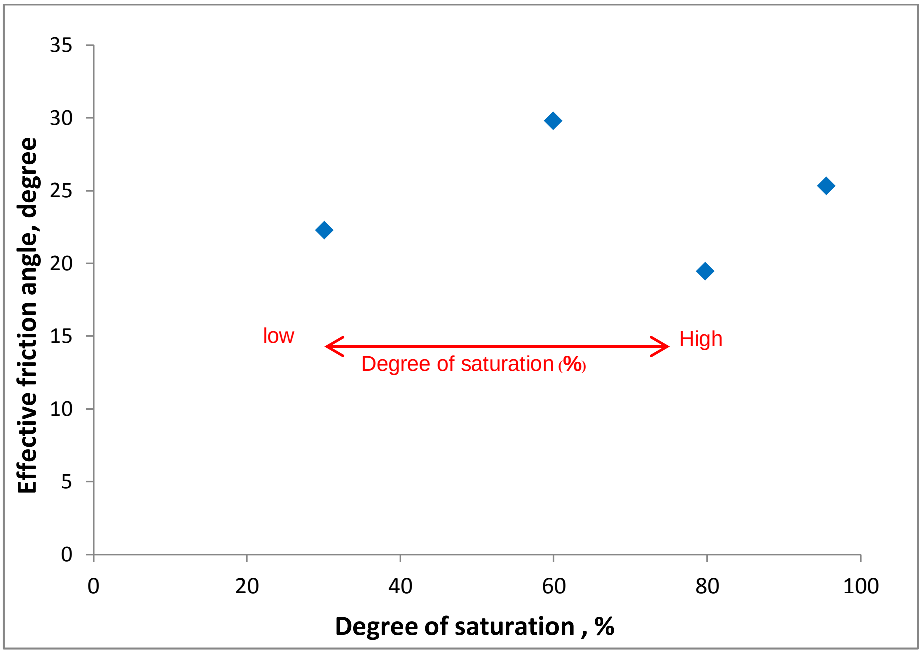

Figure 4.31. Test results relationship between effective friction angle and degree of saturation in Test pit :TP 9 (CU test).

From testing result can see that cohesion and friction angle in each Test Pit have relationship with the degree of saturation value and considering the Shear stress value, Normal stress value with degree of saturation by bringing a testing result in each Test Pit plot in system three axes. with in x axis that is Degree of saturation value , y axis that is Normal stress value and z axis that is Shear stress value shown in figure 4.32 found that Shear stress value has a tendency decreases when degree of saturation value increasing and Normal stress value decreases or Shear strength value has a tendency decreases when degree of saturation value increasing. And bringing the testing result fit the equation is plane regression shown in equation (12) and equation of each Test Pit shown in equation (13 -16)

Plane equation — Strength of soil equation

$$Z = Z_0 + aX + bY \Longleftrightarrow \tau = \tau_0 + aSr + b\sigma \qquad (12)$$

Test Pit : TP 1 $\quad \tau = 0.79874 - 0.006389(Sr) + 0.4163\,(\sigma) \quad r^2 = 0.7292 \quad (13)$

Test Pit : TP 4 $\quad \tau = 1.67107 - 0.014756(Sr) + 0.39011\,(\sigma) \quad r^2 = 0.8342 \quad (14)$

Test Pit : TP 8 $\quad \tau = 1.06757 - 0.007191(Sr) + 0.4359(\sigma) \quad r^2 = 0.8044 \quad (15)$

Test Pit : TP 9 $\quad \tau = 0.84477 - 0.005952(Sr) + 0.44872(\sigma) \quad r^2 = 0.9190 \quad (16)$

When

τ = Shear Stress, ksc

τ_0 = Constant Factor

a = Slope of the relationship between shear stress and degree of saturation

Sr = Degree of Saturation , %

b = Slope of the relationship between shear stress and normal stress

σ = Normal Stress , ksc

From plane equation of 4 Test pit can be seen that at Degree of saturation value is nearly 0.01 and Normal stress value is low. Strength of soils value from plan equation will be negative.

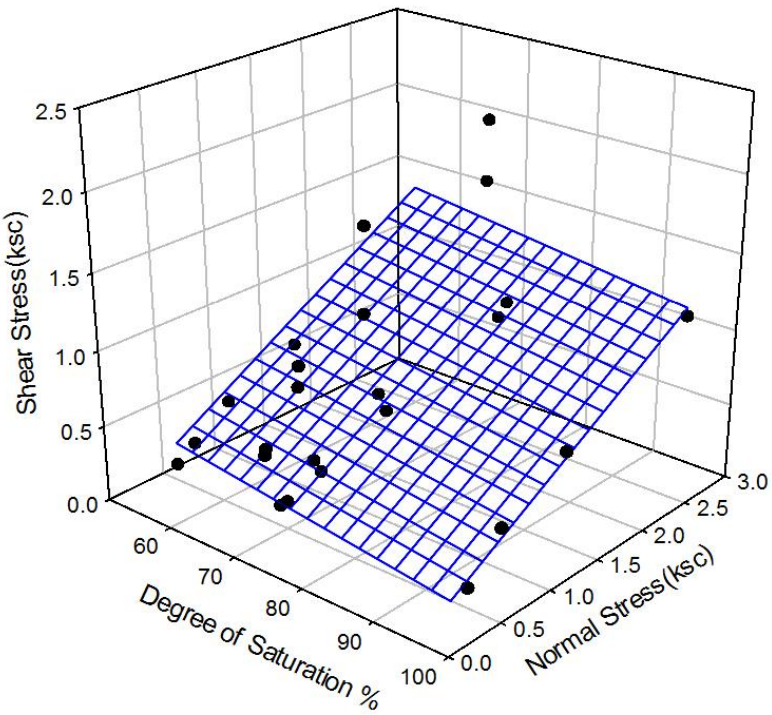

Figure 4.32. Testing result direct shear test (Multi-stage type) from TP 1.

Therefore, the consideration is made to find the plane equation appropriate with the consideration from testing result shear strength of soil at various normal stress value. After that, the transition of soil shear strength value at various degree of saturation value is compared with a degree of saturation 100 % in each test pit.

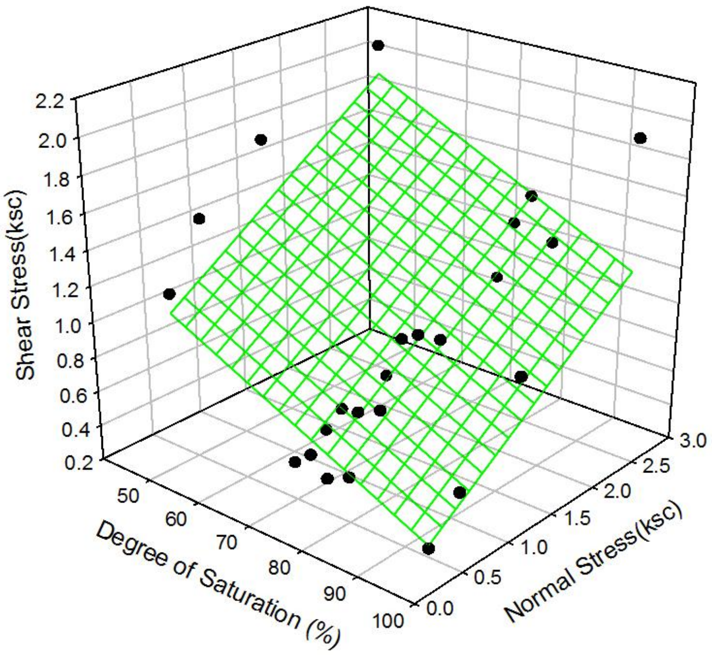

Figure 4.33. Testing result direct shear test (Multi-stage type) from TP 4.

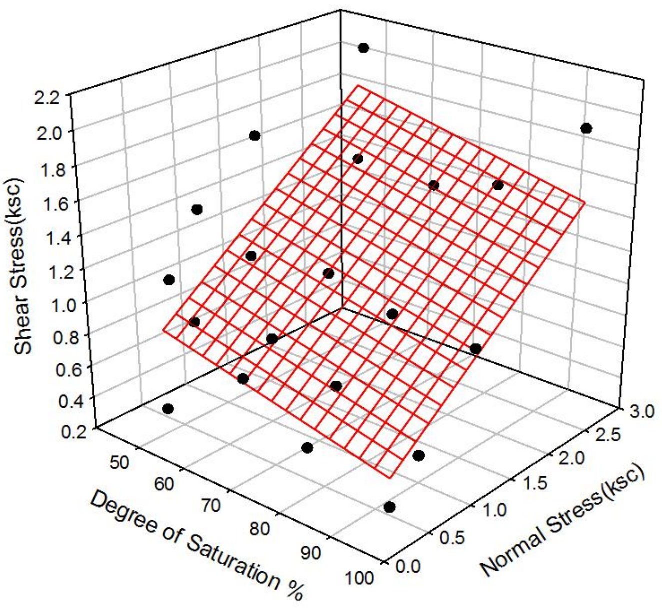

Figure 4.34. Testing result direct shear test (Multi-stage type) from TP 8.

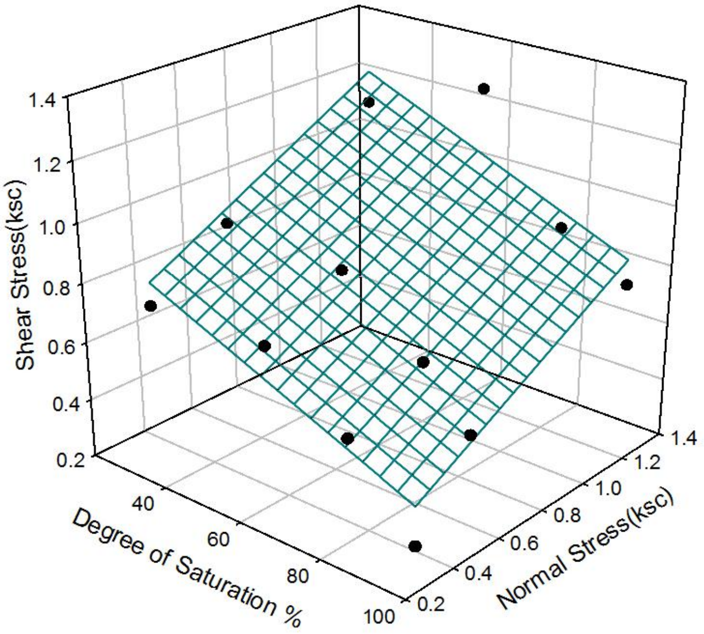

Figure 4.35. Testing result direct shear test (Multi-stage type) from TP 9.

4.8 Slope stability analysis

Moisture content in the soil affected stability the soil slope by making water pressure in the soil has changed. Water pressure herein can be divided into 2 sections. First section is affected shear strength especially in the case unsaturated soil when water pressure increases thus making shear strength decreases. Second section is affected the water level of soil slope that is ground water level will increase the pressure (uplift pressure), thus making the Factor of safety (F.S.) value decreases.

4.8.1 Slope stability analyze with Infinite slope method

In order to know the range the factor of safety value possibility for slope stability analysis has shear strength change according saturation value and the stability analysis spot including critical saturation value that making factor of safety value equal to 1. Therefore, the stability analysis in soil slope that has properties of soil and slope angle all equal by using the relationship shear strength has from testing result is needed. Slope angle of soil slope select using slope angle of soil slope equal to 25 degree and thickness of soil layer equal to 1, 2 and 3 meter. Result of the analysis obtained from this research will be the factor of safety value of soil slope that has saturation value shown in tables 4.13, 4.14 and 4.15 (consolidation drained test) and table 4.16 (consolidation undrained test).This case study with not calculated seepage condition or Groundwater condition.

The factor of safety value varies inversely with saturation value by which the range of critical saturation at approximate 90% , 80% and 70 % for thickness of soils layer , equal to 1 , 2 and 3 meter, respectively. It is meant that when the saturation value of soil higher than 70%, soil slope will begin unstable. The stability of soil slopes is also depending on the saturation value or moisture content in soil mass sure enough.

In the case for the slope stability analysis of soil slope, specific spot by using the analysis result includes the saturation value from the flow analysis which the saturation of soil slope value is unequal. Therefore, the results are the factor of safety value distribution on the soil slope and easy to consideration. Therefore, F.S. can be divided as shown in table 4.12.

Table 4.12. The range of Factor of Safety relationship with stability of soil slope (Thaijeamaree , 2003)

Range of Factor of safety	Stability of soil slope
0.2 – 0.5	Very few
0.5 – 1	Low
1- 2	Moderate
2 - 20	Very

Table 4.13. Factor of safety soil slope analysis by Infinite slope at thickness of soils layer equal to 1 , 2 and 3 meter (TP 1, consolidation drained test)

Degree of saturation	Factor of safety		
	1 meter	2 meter	3 meter
50	1.203	1.109	1.011
60	1.006	0.930	0.911
70	0.930	0.920	0.904
80	0.917	0.911	0.900
90	0.912	0.907	0.899
100	0.905	0.899	0.897

Table 4.14. Factor of safety soil slope analysis by Infinite slope at thickness of soils layer equal to 1 , 2 and 3 meter (TP 4, consolidation drained test)

Degree of saturation	Factor of safety		
	1 meter	2 meter	3 meter
50	1.142	1.042	1.039
60	0.939	0.905	0.871
70	0.898	0.867	0.857
80	0.877	0.861	0.854
90	0.860	0.848	0.844
100	0.852	0.846	0.843

Table 4.15. Factor of safety soil slope analysis by Infinite slope at thickness of soils layer equal to 1 , 2 and 3 meter (TP 8, consolidation drained test)

Degree of saturation	Factor of safety		
	1 meter	2 meter	3 meter
50	1.142	1.088	1.071
60	0.939	0.988	0.991
70	0.861	0.877	0.896
80	0.829	0.857	0.867
90	0.744	0.828	0.824
100	0.712	0.736	0.743

Table 4.16. Factor of safety soil slope analysis by Infinite slope at thickness of soils layer equal to 1 , 2 and 3 meter (TP 9, consolidation undrained test)

Degree of saturation	Factor of safety		
	1 meter	2 meter	3 meter
50	1.219	1.119	1.040
60	1.040	1.001	0.988
70	1.014	0.993	0.984
80	0.998	0.989	0.980
90	0.996	0.979	0.973
100	0.985	0.976	0.972

In addition, In order to know range the factor of safety value possibility for slope stability analysis has shear strength change according saturation value and the stability analysis spot including critical saturation value that making factor of safety value equal to 1. Therefore has the stability analysis soil slope that has properties of soil and slope angle all equal by using the relationship shear strength has from testing result. Slope angle of soil slope select using varies slope angle of soil slope equal to 20 degree, 25 degree, 35 degree and fix thickness of soil layer equal to 1 meter. Which result the analysis obtained will be the factor of safety value of soil slope that have saturation value equal to 60 % , 80 and 100 %,respectively. Shown in table 4.17 , 4.18 and 4.19 (about consolidation drained test) and table 4.20(about consolidation undrained test).This case study with not calculated seepage condition or Groundwater condition.

The factor of safety value varies inversely with saturation value by which the range of critical saturation at approximate in the range 80 % to 90 % for thickness of soils layer equal to 1 meter. Meant when the saturation value of soil higher than 80 % soil slope will begin unstable and result analysis factor of safety using by the analysis stability of soil slope with method of infinite slope found factor of safety have value decreases according to the level of degree of saturation increases when soil slope have value slope angle is greater than or equal 25 degree. Shows that the stability of soil slope depending on the saturation value or moisture content in soil mass sure enough.

Table 4.17. Factor of safety soil slope analysis by Infinite slope at slope angle equal to 20 , 25 and 35 degree (TP 1, consolidation drained test)

Degree of saturation	Factor of safety		
	20 degree	25 degree	35 degree
60	1.213	0.949	0.636
80	1.189	0.930	0.622
100	1.168	0.912	0.609

Table 4.18. Factor of safety soil slope analysis by Infinite slope at slope angle equal to 20 , 25 and 35 degree (TP 4, consolidation drained test)

Degree of saturation	Factor of safety		
	20 degree	25 degree	35 degree
60	1.207	0.946	0.638
80	1.152	0.901	0.605
100	1.102	0.861	0.575

Table 4.19. Factor of safety soil slope analysis by Infinite slope at slope angle equal to 20 , 25 and 35 degree (TP 8, consolidation drained test)

Degree of saturation	Factor of safety		
	20 degree	25 degree	35 degree
60	1.101	0.860	0.574
80	1.054	0.822	0.546
100	1.011	0.788	0.521

Table 4.20. Factor of safety soil slope analysis by Infinite slope at slope angle equal to 20 , 25 and 35 degree (TP 9, consolidation undrained test)

Degree of saturation	Factor of safety		
	20 degree	25 degree	35 degree
60	1.329	1.040	0.698
80	1.300	1.017	0.681
100	1.275	0.996	0.666

4.8.2 Slope stability analyze with limit equilibrium method

In order to know the range of factor of safety for analyzing the possibility for slope stability, shear strength change according to saturation value and the spot stability analysis include critical saturation value that making factor of safety value equal to 1. Slope stability analyses were done by KU - slope computer program developed by Kasetsart University (Isaroranit, 2001). The geometry of the mountain slope was studied to select the appropriate cross section for the analysis. The stability analysis that is so-called "Circular failure and non - circular failure " in soil slope was carried out using the relationship shear strength from testing result along resistivity surveys in the study area (figure 4.36). Results are shown and color in KU- slope program is explained in table 4.21.

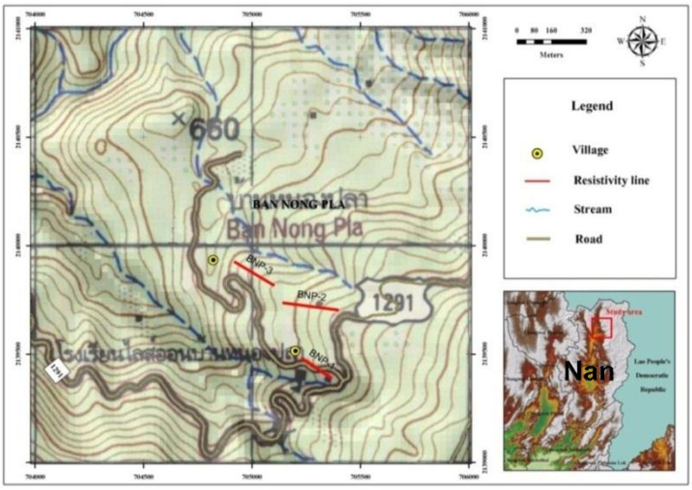

Figure 4.36. 2-D resistivity survey location in study area.

Table 4.21. Attribute of various zones based on 2-D resistivity line survey.

Number	Color symbol	Description
1	Green	Failure slip surface
2	Orange	Residual soil (Degree of weathering: Grade VI – IV)
3	Blue	The basement rock: fresh shale

Line survey at BNP – 1 (Initial water content condition) : using cohesion of shale value, friction angle value and unit weight value of shale equal to 25 t /m^2 , 60 degree and 19 t / m^3 (Irsyam et al., 2006), respectively. Cohesion of soil value, friction angle value and unit weight value of soil layer are equal to 5 t/m^2, 25.3 degrees and 1.8 t/m^3, respectively. This can be concluded that “Blue color “that is fresh shale. This zone is dominated by *in situ* shale in the profile, at the depth of 15 to 20 meters and the "Orange color" is residual soil. This zone is the result of the weathering process that changes the rock into soil and has approximate thickness average values as 2- 5 meters and “Green color” is failure slip surface of soil slope. Finally, result of the soil slope stability analysis the <u>factor of safety of BNP-1 equal to 2.32</u> this case study with not calculated seepage condition and load effect (see figure 4.37).

Line survey at BNP – 1 (Degree of saturation condition greater than or equal 90 percent) : using cohesion of shale value, friction angle value and unit weight value of shale equal to 25 t /m^2 , 55 - 60 degree and 19 t /m^3(Irsyam et al ., 2006), respectively. Cohesion of soil value, friction angle value and unit weight value of soil layer equal to 1 t /m^2, 21.58 degree and 1.8 t / m^3, respectively. This can be concluded that “Blue color “is fresh shale. This zone is dominated by *in situ* shale in the profile, at the depth of 15 to 20 meters and the "Orange color” that is residual soil. This zone is the result of the weathering process that change the rock into soil and has approximate thickness average values as 2- 5 meters and “Green color “that is failure slip surface of soil slope. Finally, result of the soil slope stability analysis <u>the factor of safety of BNP-1 equal to 1.18</u> this case study with not calculated seepage condition and load effect (see figure 4.38).

Line survey at BNP – 2 (Initial water content condition): using cohesion of shale value, friction angle value and unit weight value of shale equal to 25 t /m^{2,} 55 - 60 degree and 19 t /m^3 (Irsyam et al., 2006), respectively and using cohesion of soil value, friction angle value and unit weight value of soil layer equal to 11.9 t /m^2, 20.84 degree and 1.87 t /m^3, respectively. This can be concluded that “Blue color” is fresh shale. This zone is dominated by *in situ* shale in the profile, at the depth of 10 to 30 meters and the "Orange color" is residual soil. This zone is the result of the weathering process that changes the rock into soil and has approximate thickness average values as 3 - 9 meters and "Green color "that is failure slip surface of soil slope. Finally, result of the soil slope stability analysis <u>the factor of safety of BNP -2 equal to 2.49</u> this case study with not calculated seepage condition and load effect (see figure 4.39).

Line survey at BNP – 2 (Degree of saturation condition greater than or equal 90 percent): using cohesion of shale value, friction angle value and unit weight value of shale equal to 25 t /$m^{2,}$ 55 - 60 degree and 19 t / m^3 (Irsyam et al., 2006), respectively and using cohesion of soil value, friction angle value and unit weight value of soil layer equal to 0.9 t / m^2, 35.78 degree and 1.87 t / m^3, respectively. This can be concluded that "Blue color" is fresh shale. This zone is dominated by *in situ* shale in the profile, at the depth of 10 to 30 meters and the "Orange color" is residual soil. This zone is the result of the weathering process that changes the rock into soil and has approximate thickness average values as 3 - 9 meters and "Green color "that is failure slip surface of soil slope. Finally, result of the soil slope stability analysis <u>the factor of safety of BNP - 2 equal to 1.53</u> this case study with not calculated seepage condition and load effect (see figure 4.40).

Line survey at BNP – 3 (Initial water content condition): using cohesion of shale value, friction angle value and unit weight value of shale equal to 25 t /m^2, 55 - 60 degree and 19 t/m^3 (Irsyam et al., 2006), respectively and using cohesion of soil value, friction angle value and unit weight value of soil layer equal to 4 t/ m^2, 21.9 degree and 1.77 t / m^3, respectively. This can be concluded that "Blue color" is fresh shale. This zone is dominated by *in situ* shale in the profile, at the depth of 10 to 20 meters. and the "Orange color " is residual soil. This zone is the result of the weathering process that changes the rock into soil and has approximate thickness average values as 2 - 7 meters and "Green color "that is failure slip surface of soil slope. Finally, result of the soil slope stability analysis <u>the factor of safety of BNP -3 equal to 2.05</u> this case study with not calculated seepage condition and load effect (see figure 4.41).

Line survey at BNP – 3 (Degree of saturation condition greater than or equal 90 percent): using cohesion of shale value, friction angle value and unit weight value of shale equal to 25 t /m^{2,} 55 - 60 degree and 19 t / m^{3} (Irsyam et al., 2006), respectively and using cohesion of soil value, friction angle value and unit weight value of soil layer equal to 1.4 t / m^{2}, 19.79 degree and 1.77 t / m^{3}, respectively. This can be concluded that "Blue color" is fresh shale. This zone is dominated by *in situ* shale in the profile, at the depth of 10 to 20 meters and the "Orange color" is residual soil. This zone is the result of the weathering process that changes the rock into soil and has approximate thickness average values as 2 - 7 meters and "Green color" is failure slip surface of soil slope. Finally, result of the soil slope stability analysis <u>the factor of safety of BNP - 3 equal to 1.26</u> this case study with not calculated seepage condition and load effect (see figure 4.42).

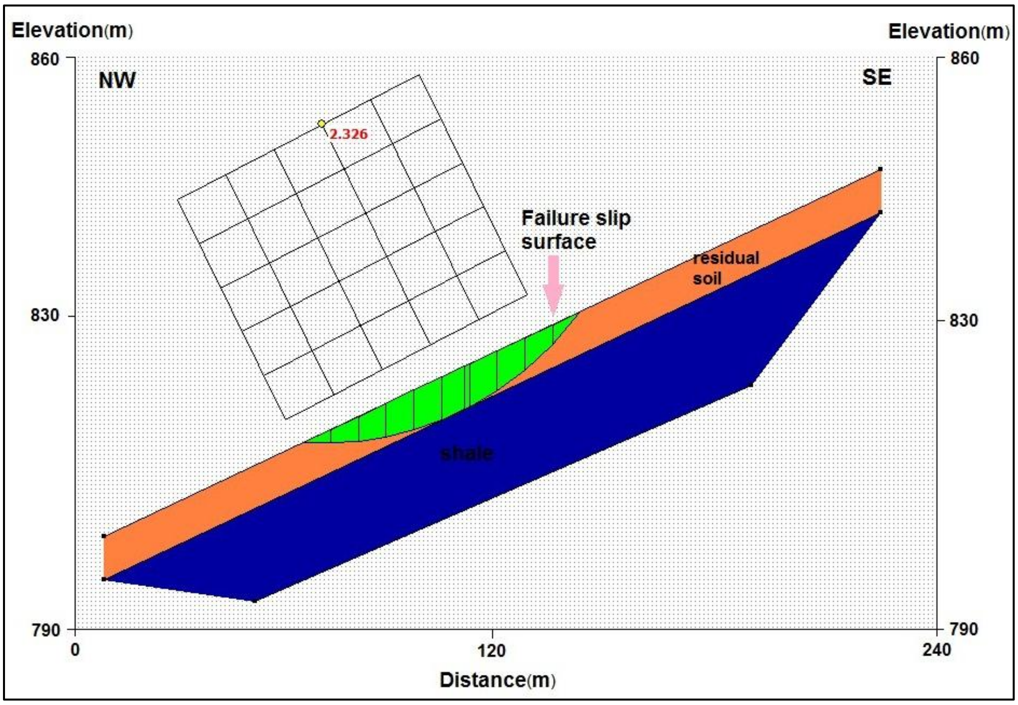

Figure 4.37. Factor of safety soil slope analysis by KU - slope at initial water content condition (BNP- 1).

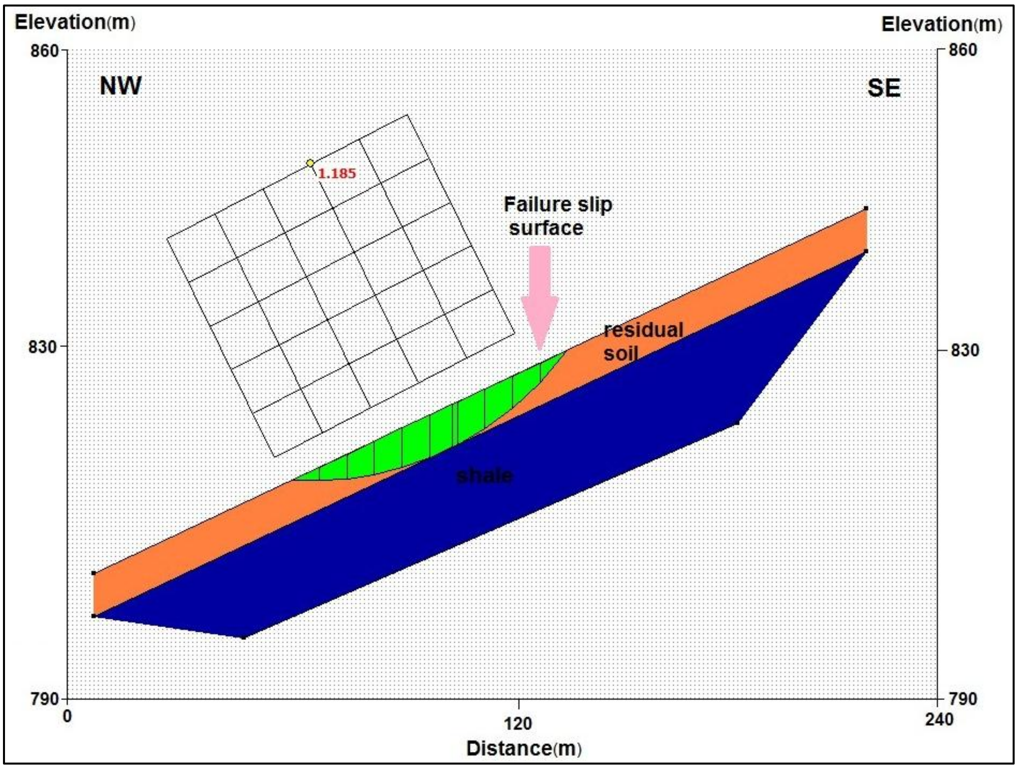

Figure 4.38. Factor of safety soil slope analysis by KU - slope at degree of saturation condition greater than or equal 90 percent (BNP- 1).

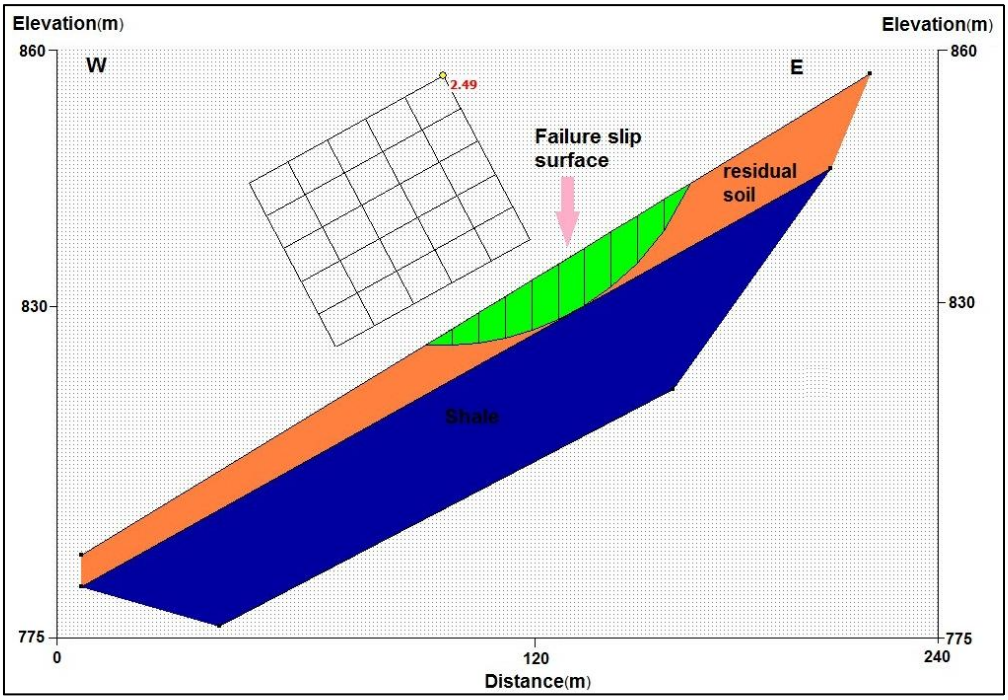

Figure 4.39. Factor of safety soil slope analysis by KU - slope at initial water content condition (BNP- 2).

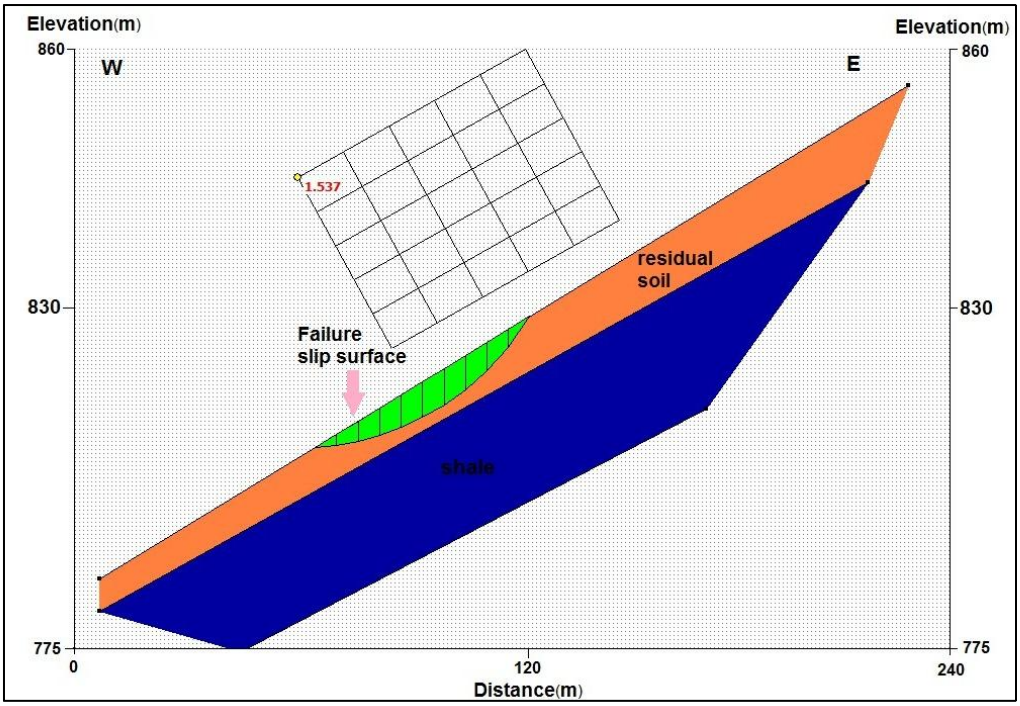

Figure 4.40. Factor of safety soil slope analysis by KU - slope at degree of saturation condition greater than or equal 90 percent (BNP- 2).

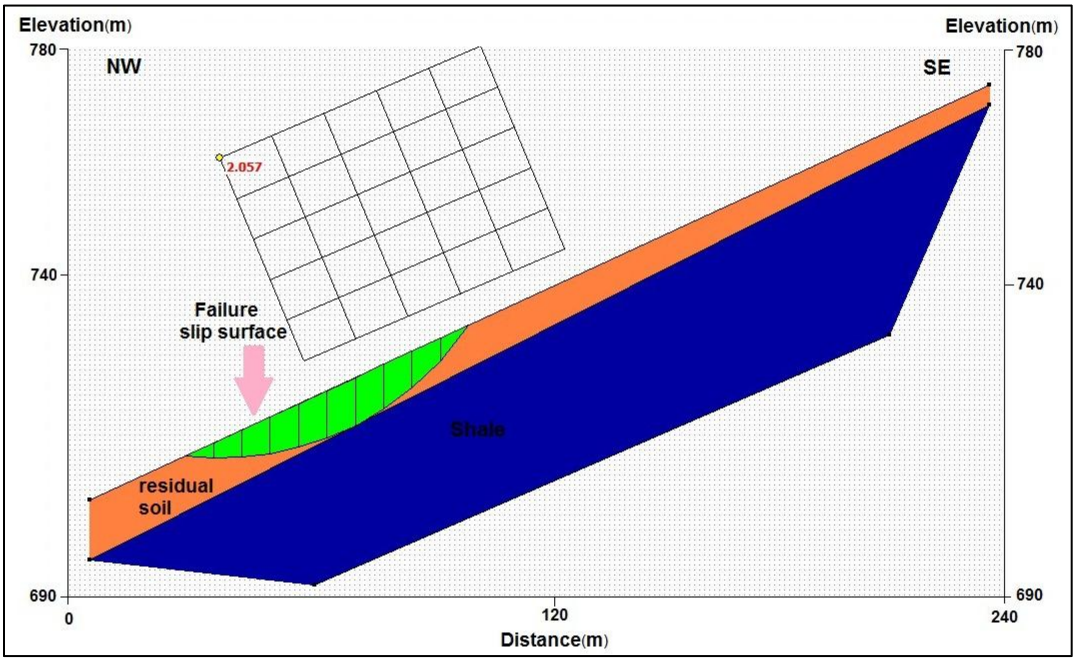

Figure 4.41. Factor of safety soil slope analysis by KU - slope at initial water content condition (BNP- 3).

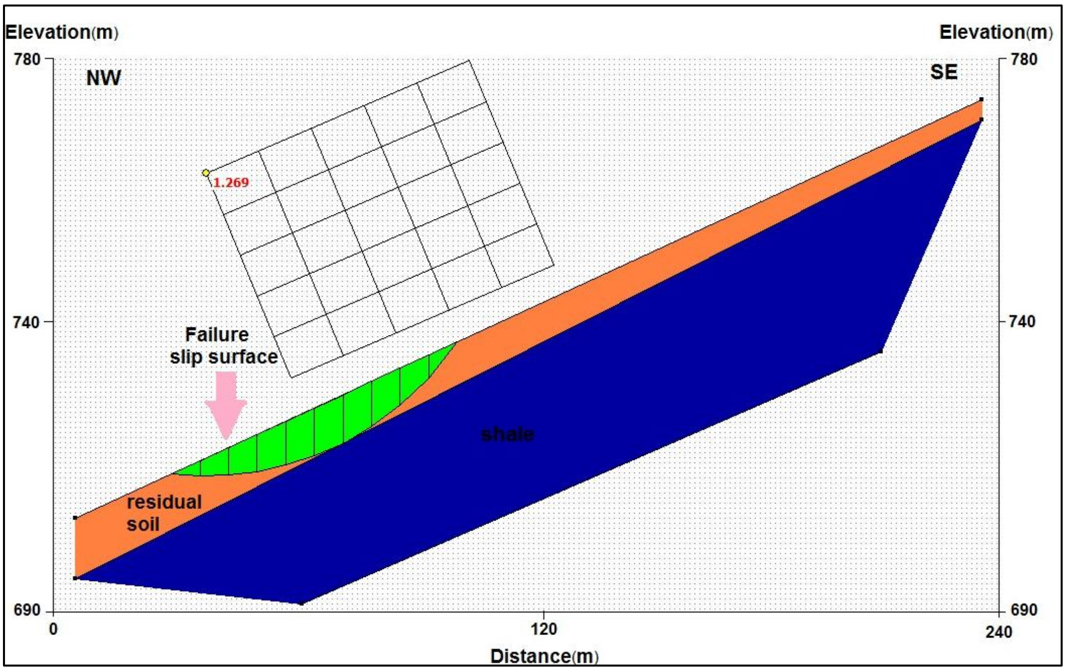

Figure 4.42. Factor of safety soil slope analysis by KU - slope at degree of saturation condition equal or more than 90 percent (BNP- 3).

The factor of safety value varies inversely with saturation value by which the range of critical saturation at approximate greater than or equal 90 % for thickness of soils layer greater than or equal to 5 meter. It means when the saturation value of soil higher than 90 % soil slope will begin unstable and result analysis factor of safety using by the analysis stability of soil slope with the KU – Slope program. It is found that factor of safety has value decreases according to the level of degree of saturation increases. Also, it is suggested that the stability of soil slopes depends on the saturation value or moisture content in soil mass.

Chapter V

DISCUSSIONS

In this chapter, the main discussion will be focused on how the results described in the preceding chapters can relate the importance of shear strength of soil that decreases when the moisture content is changing, relationship between factor of safety that decreases when slope angle increases and the factor of safety that decreases when thickness of soil increases. Based on three resistivity lines survey, the thickness of soil overlying on rock basement shows its importance for calculating the volumetric of possible collapse. Finally, the comparison between results from this research with the work of Akkrawintawong (2008) that studied about landslides disaster at Nan Province in northern Thailand will be discussed.

5.1 Electrical resistivity surveying method (2-D)

2-D resistivity surveys were assigned in this research aiming to find the thickness of soil layer and unconsolidated sediment overlaying on weathered bed rock (estimation of the depth of soil layer). The 3 lines electrical resistivity survey (2-D) in the study area revealed that soils layer owns approximate thickness in range between 2 -9 meters. Lines survey BNP- 2 shows thickness of soil between 3 - 9 meters to which this range in thickness owns an opportunity to cause the landslide. During heavy rain, this soil will be more opportunity to slide than a normal situation because of fine-grained of residual soils. The increases in water volume in soil may result the decrease of strength in soil.

5.2 Physical properties of residual soil

This section will discuss on the result of analysis in physical properties of weathered residual soil from parent rock with special attention to study the feasibility and the opportunity of landslide. As a result, residual soil from weathered parent rocks such as shale, siltstone and sandstone occurred in this study area includes high percentage of fine grained texture(see figure 5.1 and 5.2) and have characteristic of distribution is Gap grade and Uniform grade.

Most of residual soils in the study area owns the low plasticity (LL< 47 % and PI = 15.58 %) shows the test results in plasticity chart (see figure 5.3) indicates kaolinite and clay mineral in soil and can be classified by Unified Soil Classification System including ML- CL, CL- ML, SM. which is meant that the toughness properties of soil will be changed whenever the moisture content increases, event just a little increase (Hazelton, P. and Murphy, B. 2007). This soil type is risky to cause the landslide.

Soralump (2008) found that most of soil types including CL and CH uniformly classified by USCS contains high percentage of grained texture that the test results in plasticity chart (see figure 5.4) indicates illite and kaolinite mineral in soil. This soil type is able to lose a lot of strength when it was taken soak (SRI = 1.581). Therefore, this soil type is risky to cause the landslide. In addition, Soralump and Thowiwat (2009) also found that residual soils from clastic sedimentary rock such as shale, sandstone, mudstone (Group 5) which is well grade and contains high percentage of fine grained texture. They can be classified by USCS as ML, CL and SM (see figures 5.5, 5.6). Likewise, this soil type is able to lose a lot of strength when it was taken soak (SRI = 1.581). Therefore, this soil type is risky to cause the landslide.

Tepparnich and Jotisankasa (2010) suggested that residual soil form sedimentary rock such as shale and mudstone is commonly composed of high percentage of fine grained texture which can be classified by Unified Soil classification systems included ML and SM and have the low plasticity value (LL= 41.75 % and PI = 11.50 %) (see figure 5.7). This means that the toughness properties of soil will be changed whenever the moisture content increases, even just a little increase. This soil type is also risky to cause the landslide.

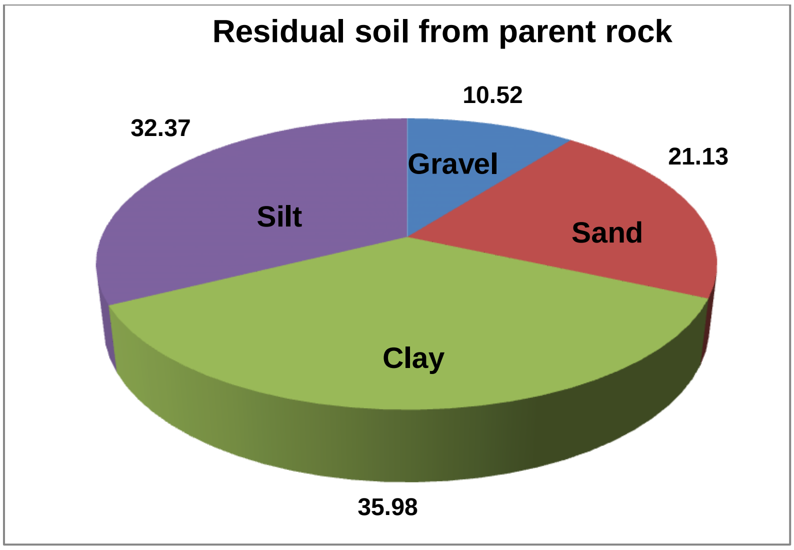

Figure 5.1.Results the percentage of grain size distribution in this research.

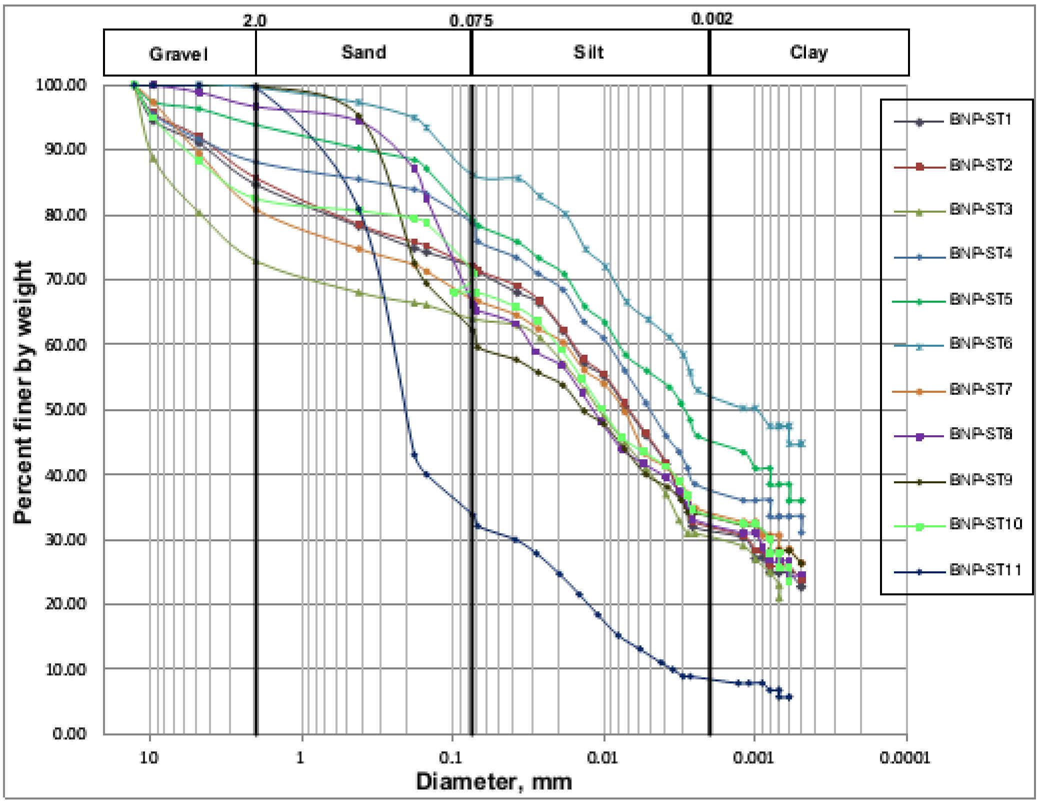

Figure 5.2. Grain size distribution of residual soil from weathered parent rock in this research.

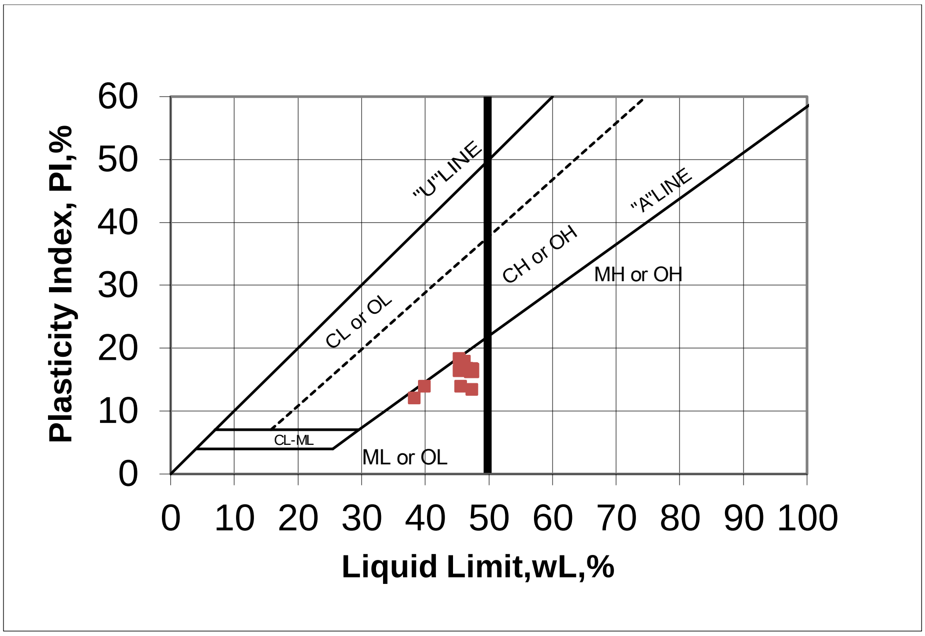

Figure 5.3. Test results plasticity chart in this research.

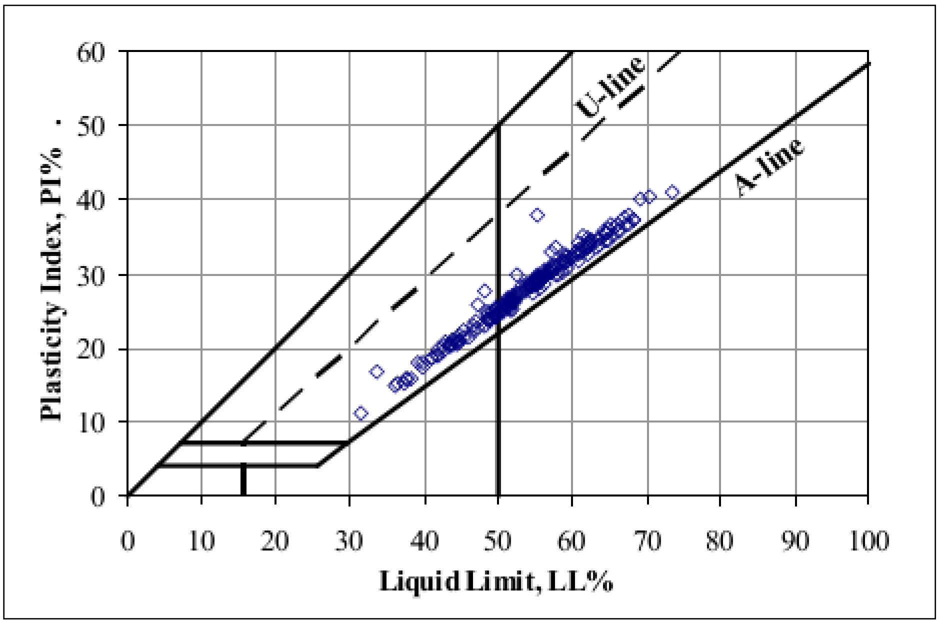

Figure 5.4. Test results plasticity chart studied by Soralump (2008).

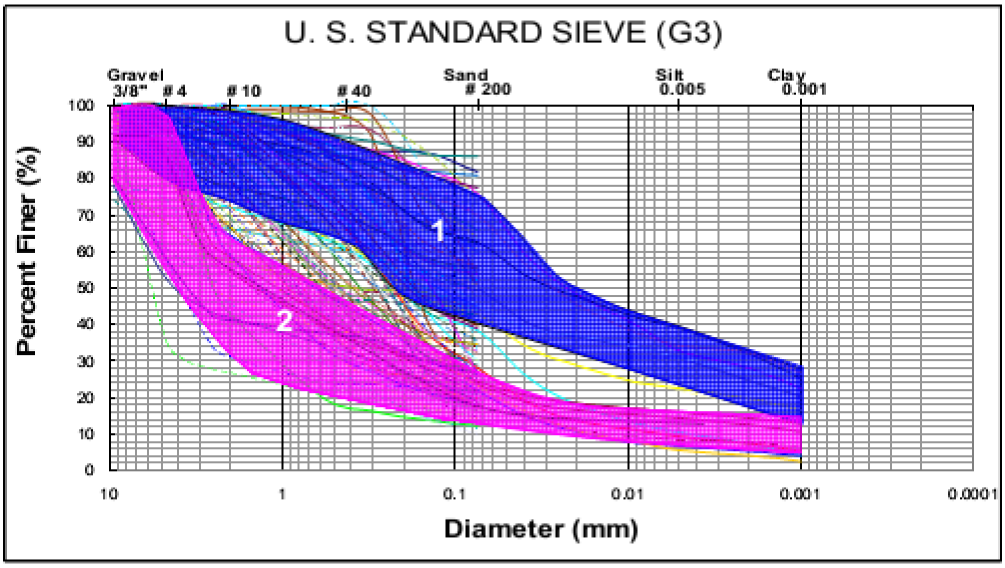

Figure 5.5. Test results grain size distribution studied by Soralump and Thowiwat (2009).

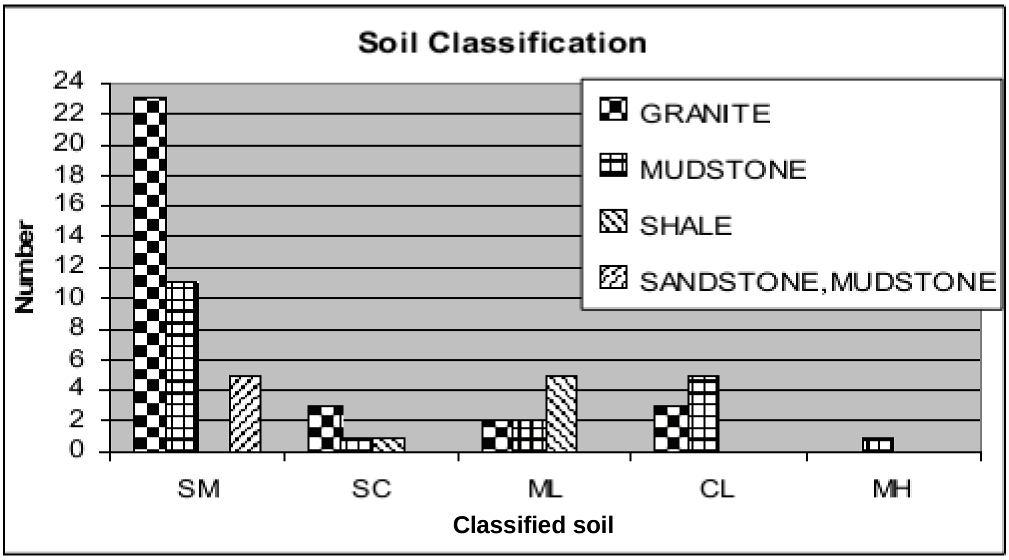

Figure 5.6. Soil classification of residual soil studied by Soralump and Thowiwat (2009).

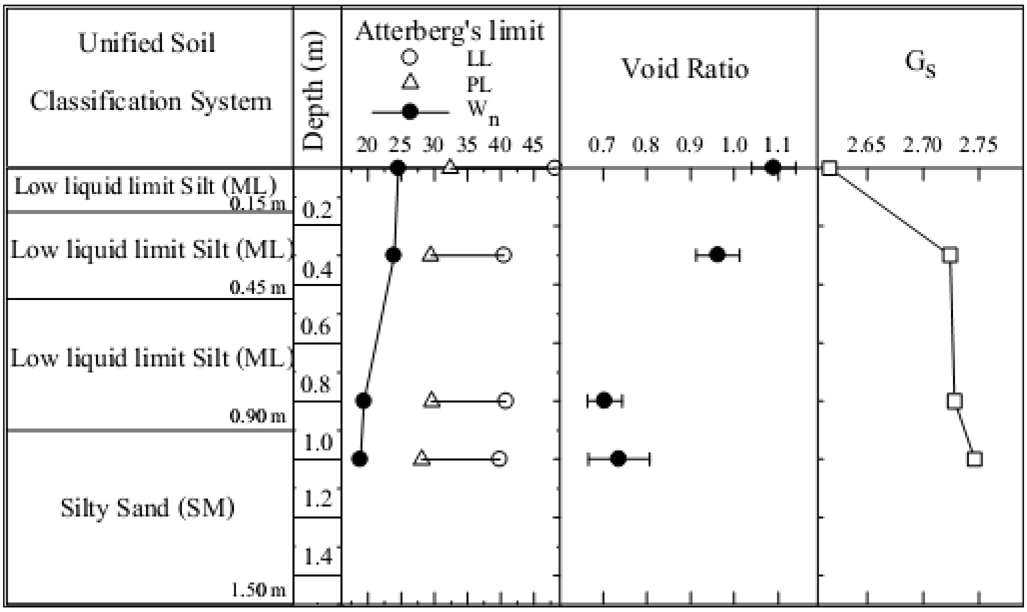

Figure 5.7. Physical properties of soil layer according to thickness studied by Tepparnich and Jotisankasa (2010).

5.3 Engineering properties of residual soil

Analysis of engineering properties of residual soil from parent rock was carried out for evaluate the possibility and opportunity for the cause of landslide. Basically, the shear strength of soil in natural condition decreases when the moisture content increases. As a result of shear strength for Consolidation drained testing (amount 8 test pits); the cohesion of soil values ranges from 0.096 to 1.196 ksc and angle of internal friction is between 11.51 to 35.78 degree. Consolidation undrained testing (amount 1 test pits) ranges from 0.18 to 0.617 ksc and angle of internal friction is between 19.45 to 29.80 degree.

Therefore, the understanding of soil shearing strength in natural condition and various control moisture content condition is necessary for analyzing soil stability problems. In addition, the shear strength parameter (i.e., cohesion and friction angle) is able to analyze the model of stability of soil slopes in order to find the plane of failure and the safety factor of soil slope.

5.4 Total cohesion of soil and degree of saturation

This research found that the properties of soil shear strength decreases when moisture content increases (see figure 5.8). The consolidation drained testing of cohesion of soil ranges from 0.096 to 1.196 ksc of residual soil from shale and siltstone and 0.18 to 0.617 ksc from sandstone.

Therefore, total cohesion of soil decreases when the volume moisture content in soil increases. Moisture content is an important factor affecting the shear strength of the soil. Soil mass with dry conditions or low moisture content will own higher soil shear strength. Soil shear strength will decrease when moisture content in soil increases.

The volume of water increase will disturb surface tension in soil making cohesion of soil decreases and the volume of water then, increases. A friction between soil particles, then, reduces to which soil shear strength will decrease and impact directly to the stability of soil slope.

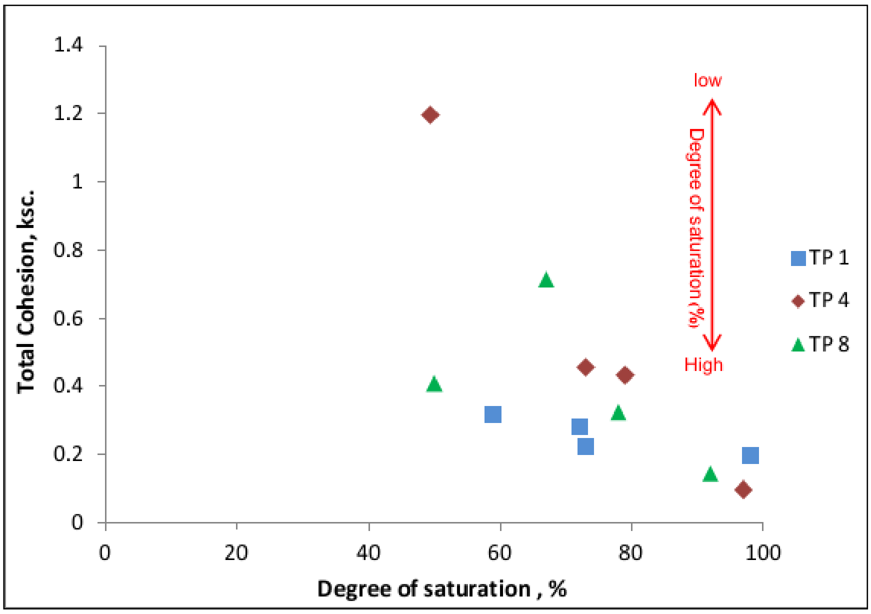

Figure 5.8. Test results total cohesion and degree of saturation (Test Pit : TP 1, TP 4 and TP 8) in this research.

5.5 Shear strength of soil and volume moisture content in soil mass

Shear strength of soil decreases when the moisture content changes. The results showed that soil shear strength rate decreases in the range 72.11 % to 98.78 %, 83 % to 91.20 % and 79.6% to 95.6% for residual soil from shale, siltstone and sandstone, respectively (see figure 5.9, 5.10 and 5.11).

Strength of soil will decrease when the moisture content in soil mass increases continuously. Therefore, it can be concluded that natural soil slope in the study area can be failure when the moisture content between soil particle increases leading to the decrease in strength of soil.

Soralump and Kunsuwan (2006) found that when soil is in saturated condition, shear strength commonly decreases from natural condition up to more than 50 % (SRI = 1.581). Result of test above is consistent with testing result of direct shear test (multi-stage type) which is found that shear strength of soil decreases when the moisture content change or when soil is in saturated condition.

Soralump et al. (2007) also found that residual soil or weathered rock from sandstone and siltstone will own shear strength which will decrease more than 50 % when soil becomes saturated. Testing result as above mentioned is consistent with testing result direct shear test (multi-stage type) on which shear strength of soil will decrease when soil is in saturated condition. This soil type is risky to cause the landslide. Later on, Soralump and Thowiwat (2009) concluded that when soil is in saturated condition leading to the decrease in shear strength (sedimentary rock: Group 5). They noticed that the friction angle values (ϕ') have relatively constant, but the cohesion of soil (c') has a lot of decrease when degree of saturation increases(see figure 5.12). Furthermore, Found that Shear strength of soil decreases when the moisture content changes. the results showed that soil shear strength rate decreases in the range 75 % to 90 % (see figure 5.13) for residual soil from parent rock (Group 5, 6)

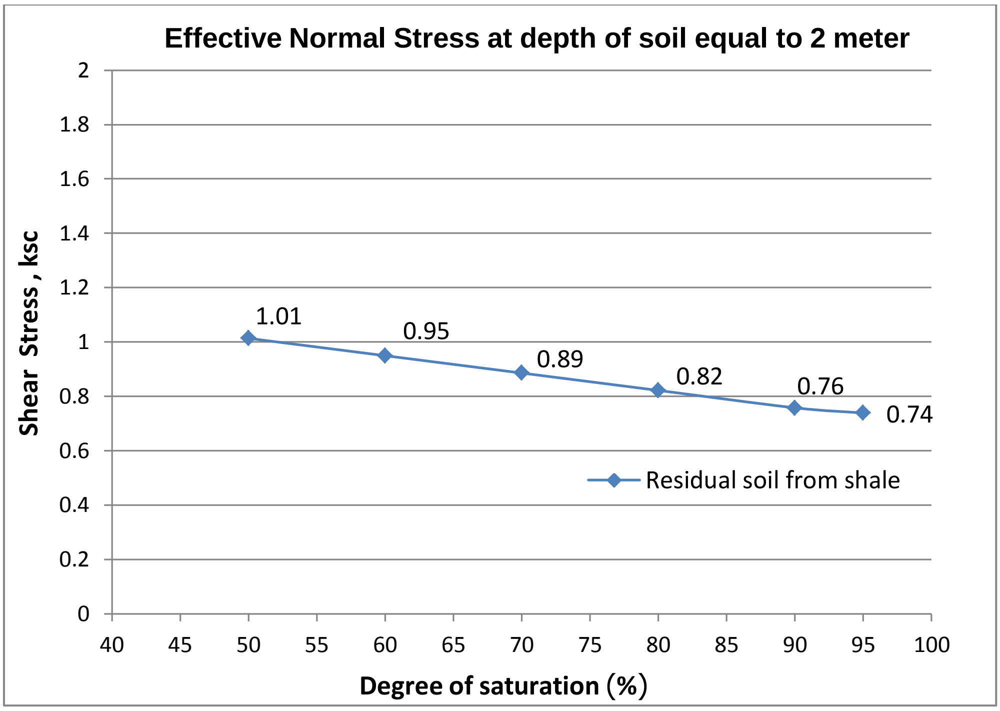

Figure 5.9. Testing results KU-MDS test of residual soil from shale in this research

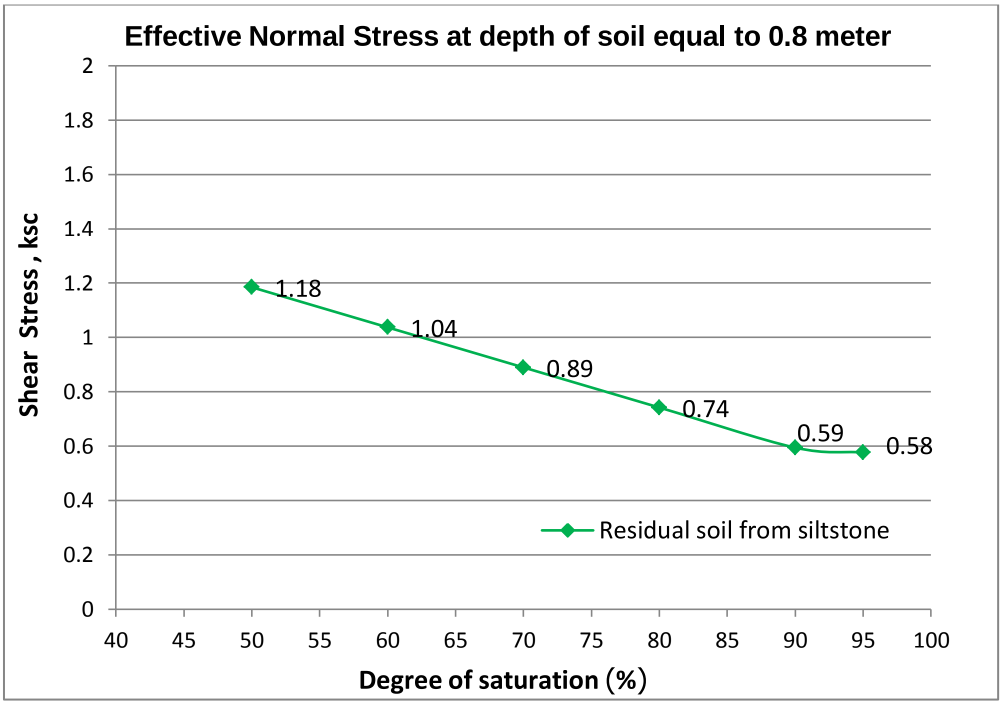

Figure 5.10. Testing results KU-MDS test of residual soil from siltstone in this research

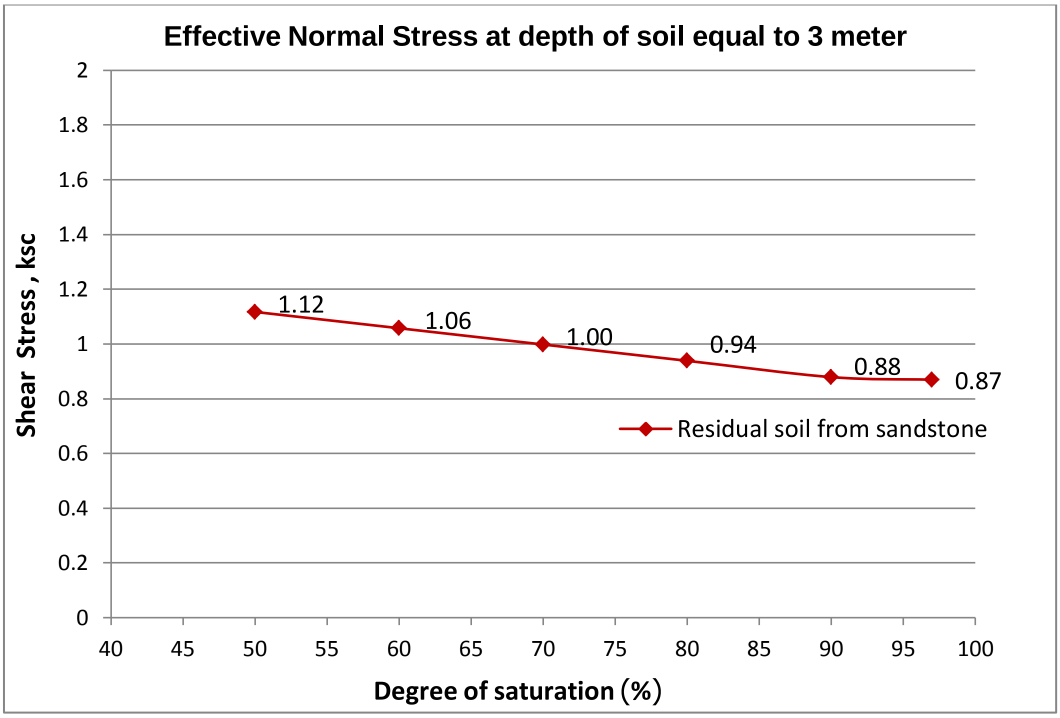

Figure 5.11. Testing results KU-MDS test of residual soil from sandstone in this research

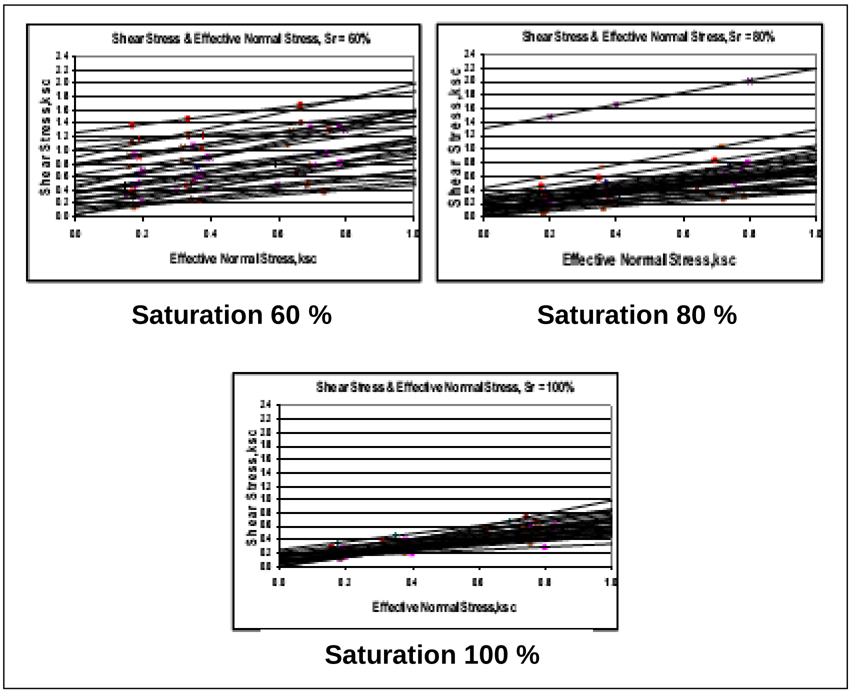

Figure 5.12. Testing results KU-MDS test of residual soil from sedimentary rock (Group5) studied by Soralump and Thowiwat (2009).

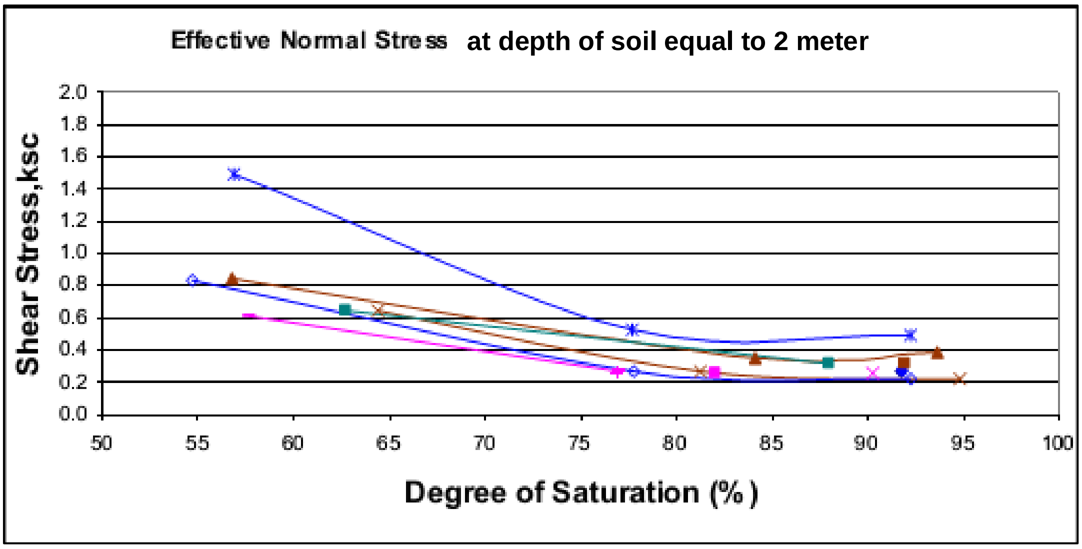

Figure 5.13. Testing results KU-MDS test of residual soil from parent rock (Group 5, 6) studied by Soralump and Thowiwat (2009).

5.6 Factor of safety and slope angle

The behavior for soil slope failure at upstream areas in Huai Nam Puea river basin, Ban Nong Pla village, Amphoe Chiang Klang, Changwat Nan will be discussed here. The undisturbed soil samples were tested to obtain soil shear strengths at the various control water contents. The results showed that the failure slope can be occurred where slope angle of this area is greater than or equal 25 degree (see figure 5.14). From the analysis stability of soil slope using the analysis in stability of soil slope with method of infinite slope found that the values slope angle of soil slope caused to the failure is between values as 25- 45 degree.

Soralump et al. (2010) recognized that the factor of safety of cut slope is lower than the natural slope. The critical slope angle that the cutting may trigger the landslide is at 17.1 degree which is corresponded to FS equal to 1.3. Therefore, any construction making slope angle greater than 17.1 degree (see figure 5.15), may cause the failure slope.

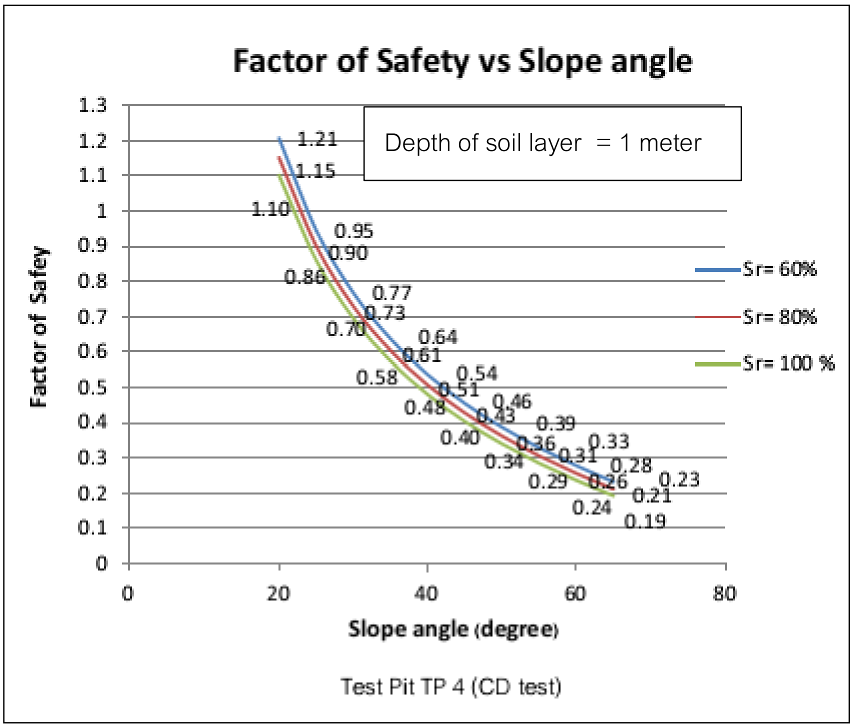

Figure 5.14. Factor of safety of natural slope Test pit : TP4 in this research.

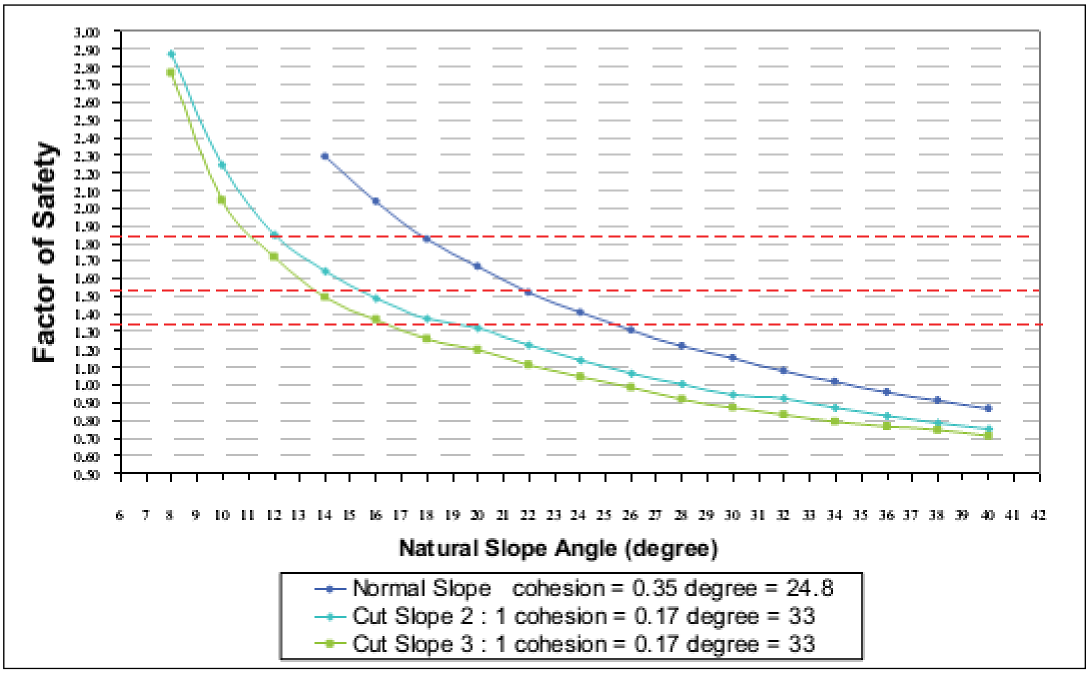

Figure 5.15. Factor of safety of cut slope and natural slope studied by Soralump et al (2010).

5.7 Factor of safety and thickness of soil layer

The analysis in stability of soil slope by method of infinite slope showed that the factor of safety decreases when soil slope reaches a critical thickness of soil layer which causing the failure is greater than or equal 1 meter according to the value slope angle under the degree of saturation level. In this area, the natural degree of saturation is ranging between 60-70% that making factor of safety becomes greater than 1. However, if the degree of saturation is greater than or equal 80%, the factor of safety soil slope will be less than 1, leading to the slope failure (see figure 5.16).

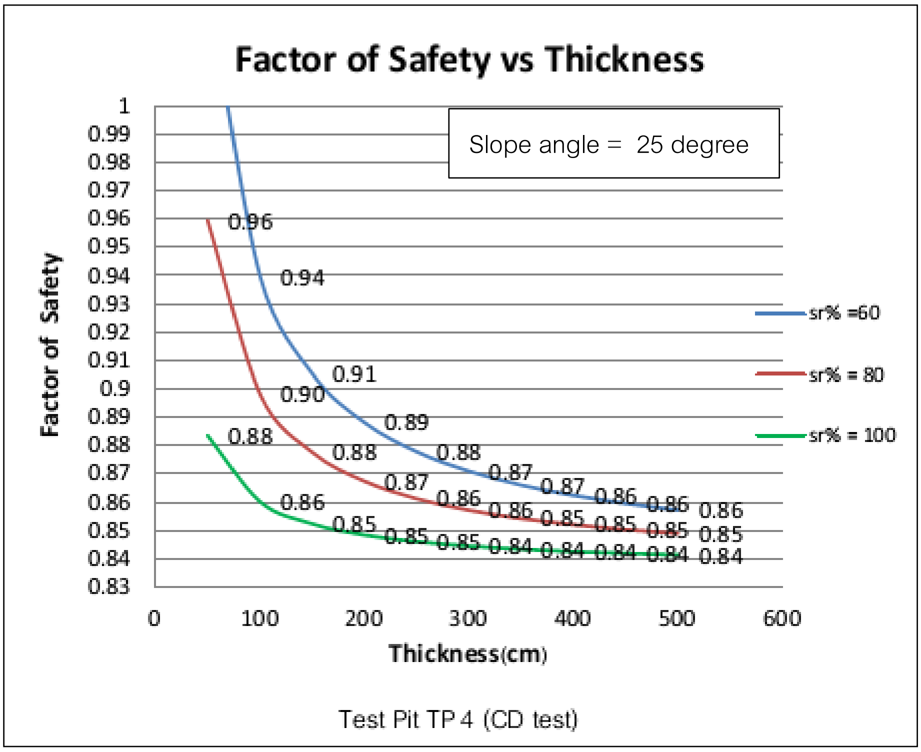

Figure 5.16. Factor of safety of natural slope at Test pit : TP4 in this research.

5.8 The comparison of this research with previous studied

This research studies the behavior for landslides on upstream areas in Ban Nong Pla village, Amphoe Chiang Klang, Changwat Nan and was focused on the analysis in engineering and physical properties of residual soil derived from parent rocks. Based on the physical properties, this research can be concluded that residual soils in the study area is characterized by silty sand soil, silty clay soil and clayey soil including fine grained soil that are qualified to poor drainage. This condition is favored for soil adsorption when raining. Furthermore, residual soils in the study area is low plasticity (LL< 47 %) that makes toughness properties of soil change when moisture content increases event just a little. Engineering properties of residual soil from parent rock can be concluded that the soil shear strength will have high decrease rate in the range of 72.11 to 98.78%, 83 to 91.20% and 79.6 to 95.6% for residual soil from shale, siltstone and sandstone, respectively. These values indicated that strength of soil will become high decreases when the moisture content in soil mass increases. Therefore, it can be concluded that when raining leading to the increase of the absorption of water, this may result in decrease of strength of soil and soil slope stability can decrease causing landslide.

Comparatively, Akkrawintawong (2008) studied about landslide disasters at Nan province in northern Thailand after a series of landslides occurred in August 23, 2006 that damaged properties and lives. As a result, the conclusion was made that the open-forest areas dominated by the steep slopes, sandstone, weathered granitic rock and tuffaceous rocks, proximal long faults/fracture, are being prone to landslide hazard. Subsequently, degrees of landslide hazard were expressed as very low to very high hazard levels. The landslide hazard map also indicated that high- to very high- level hazard areas are composed of the mountainous areas in Chalerm Prakiat, Song Khwae, Chiang Klang, Thung Chang, and Pua Districts. Therefore, result from this research is consistent with the work done by Akkrawinwong (2008).

Chapter VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The study of engineering geology regarding landslide from Ban Nong Pla village, Amphoe Chiang Klang, Changwat Nan can be summarized as follows:

1. Soils in the study area are mainly the weathering product from clastic sedimentary rocks including shale, siltstone and sandstone.

2. Most of residual soils in the study area are composed of silty sand soil, silty clay soil and clayey fine soil which can be qualified to poor drained soil. This soil character is able to easily adsorb rain water, then results in the decrease of shear strength leading to cause the landslide.

3. Most of residual soils in the study area is characterized by low plasticity (LL < 47 % and PI = 15.58 %). This means the toughness properties of soil can change when moisture content increases, even just a little increase. This is also risk for landslide.

4. Result from 3 lines electrical resistivity survey (2-D) a shows that soils layer(overburden) have approximate thickness of 2-9 meters. Survey line BNP- 2 reveals thickness of soil ranging from 3 -9 meters. Soils in this area are characterized mostly as fine-grained materials which is sensitive for sliding whenever water is induced

5. Properties of shear strength from consolidation drained testing (amount 8 test pits) are summarized. The cohesion of soil ranges from 0.096 to 1.196 ksc and angle of internal friction is between 11.51 to 35.78 degrees. Consolidation undrained testing (amount 1 test pits) shows that the cohesion of soil ranges from 0.18 to 0.617 ksc and

angle of internal friction is between 19.45 to 29.80 degrees. The properties of the soil shear strength decreases when the moisture content changes. The soil shear strength will reach the rate of high decreases in the range of 72.11 % to 98.78 %, 83 % to 91.20 % and 79.6% to 95.6% for residual soil from shale, siltstone and sandstone, respectively. Strength of soil will highly decreases when the moisture content in soil increases. Therefore, the natural soil slope in the study area can be stable when the moisture content in soil level is equal one but when the moisture content between soil particle increases, strength of soil can be decreased resulting in soil strength decreases.

6. Testing results shear strength with degree of saturation will have relationship of shear strength in the form of the equation : $\tau = \tau_0 + aSr + b\sigma$ (τ = Shear Stress, ksc ., τ_0 = Constant Factor ., a = Slope of the relationship between shear stress and degree of saturation ., Sr = Degree of Saturation ., b = Slope of the relationship between shear stress and normal stress ., σ = Normal Stress , ksc) test results are as follows : TP 1 : τ = 0.79874 - 0.006389(Sr)+ 0.4163 (σ) , TP 4 : τ = 1.67107 - 0.014756(Sr)+ 0.39011 (σ), TP 8 : τ = 1.06757 - 0.007191(Sr)+ 0.4359(σ), TP 9 : τ = 0.84477 - 0.005952(Sr)+ 0.44872(σ).

7. The analysis in stability of soil slope by method of infinite slope found that the angle of soil slope that will be caused to the failure is between 25- 45 degrees.

8. Result of the analysis in factor of safety shows that the factor of safety will decrease depending on the level of degree of saturation increases when soil slope has the slope angle greater than or equal 25 degrees. In addition, factor of safety have value decreases when critical thickness of soil layer caused to the failure is greater than or equal 1 meter according to the slope angle under the degree of saturation level.

9. Factors affecting the analysis stability of soil slope include rainfall, physical properties of soil, slope angle of soil, soil shear strength. Rain water is commonly the main factor inducing the stability of soil slope.

10. The practical way to prevent and protect landslide in the study area includes 1) the soil slope adjustment or increase water drain system to prevent soil erosion, 2) plantation to cover soil, and 3) using covered soil mesh in appropriate area.

6.2 Recommendations

The recommendation for the future investigation, monitoring and testing should be included as follows.

1. Additional testing of engineering geology in the field should be carried out in order to know the soil properties in natural condition. Additional tests include in-situ permeability test, in-situ direct shear test and standard penetration test (SPT). These additional tests will lead to the calculation of the other engineering parameters.

2. The equipment for collecting soil samples should be developed. The equipment should design to avoid sample loss during the pull out step.

3. This research found that the measurement of matric suction should be carried out together with the unsaturated test for soil strength and infiltration analyses. The permeability of soil used in analysis should be unsaturated permeability values.

4. Due to long time testing, data logger should be carried out at the same time together with direct shear test (Multi-stage type) or consolidated drained test.

5. The landslide warning for soil slope failure by using rainfall as indicator should be done by continued hourly collection of rainfall.

6. Data collection on grain size and grain distribution from the study area should be carried out both along the stream and floodplain. This is for the prediction in the area where there is high possibility to affect from rock slide.

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APPENDICES

APPENDIX A

The Equipment for collect soil sample and Method for collect sample

The Sample Collecting

The apparatus for collect soil sample in the field for testing strength value of soil sample of natural soil slope will must give a sample has condition can be representative of natural soil slope. By sample must is Undisturbed soil sample with has equipment and method as follows

The equipment and method for collect sample

The equipment for collect sample such as " Block Sample (ASTM D 4220)

The equipment for collect sample or Cylinder for collect soil sample in the field

1. Cylinder for collect sample
2. PVC Cylinder
3. The equipment will use in the packing for cushioning

Characteristic of the apparatus for collect sample design for collect Undisturbed soil sample similar the collect soil sample with Thin Walled Sampler by have

- Internal diameter size 2.5 inches

- Area Ratio ; $A_r = \dfrac{D_o^2 - D_i^2}{D_i^2} x100$

D_o^2 (Outside Diameter of Tube) = $6.90^2 = 47.6100$ cm^2

D_i^2 (Intside Diameter of Tube) = $6.35^2 = 40.3225$ cm^2

$$A_r = \left(\frac{47.6100 - 40.3225}{40.3225} x100 \right)$$

$$= 18.073 \%$$

- Contain the PVC cylinder that has diameter pass in 2.5 inches , thick 2 inches as shown in Appendix figure A1


Appendix figure A1 The equipment for collecting sample in the field

The Method for Collect Sample

Can divide 2 sections that is The collect with cylinder for collect sample in the field and The collect sample with Block sample.

The collect with cylinder for collect sample

1. Preparation the area for collect sample by dig to the require level for collect sample.
2. Preparation a cylinder for collect sample and PVC cylinder by applying vaseline, Measure the size and weighed the PVC cylinder Install number at PVC cylinder.
3. Put position preferred collect sample using a trowel slice off a soil top provided smooth regularly.
4. Put a Styrofoam beads into cylinder for reinvigorate the samples don't provided push up faster until losing quality.

5. Strike sample cylinder by divide about 3 section for take a styrofoam beads leave collect sample cylinder.
6. When strike sample cylinder to the required level for collect sample. Lead a steel (shape L) for bowl a collect sample cylinder in order that a soil at muzzle the sample cylinder absence out.
7. Pull a collect sample cylinder.
8. Put a PVC cylinder that has a soil sample to trim. Then led to weighed for find the density of soil value.
9. Scrape a soil at muzzle the PVC cylinder about 0.5 cm both two sides.
10. Drip a liquid paraffin both two sides order to prevent extinction moisture content of sample.
11. Wrapping a transparent plastic and plastic cushioning.
12. Contain a sample into foam box.

Appendix figure A2 The study area for collecting soil sample

<u>Appendix figure A3</u> Step the collecting soil sample(continue)

<u>Appendix figure A3</u>   Step the collecting soil sample (continue)

<u>Appendix figure A3</u>   led to weighed and Drip a liquid paraffin

APPENDIX B

Testing Results Direct Shear Test

Testing Results Direct Shear Test

From testing the strength of soil with testing method " Multi-stage direct shear test " in each various Test Pit. Summary testing result as shown in Appendix table B1,B2 and Appendix figure B1

Appendix table B1 : Testing results Multi-stage direct shear of Undisturbed sample at natural moisture content conditions (consolidation drained test)

Test pit	WC %	Depth(m)	Degree of weathering	c' (ksc.)	ϕ' (Degree)
TP 1	22.02	2.0	Grade V	0.223	14.77
TP 2	23.75	2.5	Grade V	0.315	20.47
TP 3	22.85	2.5	Grade V	0.501	25.30
TP 4	33.04	0.8	Grade VI	0.433	23.08
TP 5	27.57	0.6	Grade V	0.455	11.51
TP 6	26.87	1.0	Grade VI	0.525	17.82
TP 7	19.98	0.8	Grade VI	0.714	15.17
TP 8	15.81	1.2	Grade VI	0.408	21.90

Appendix table B1(continue) : Testing results Multi-stage direct shear of Undisturbed sample at natural moisture content conditions (consolidation undrained test)

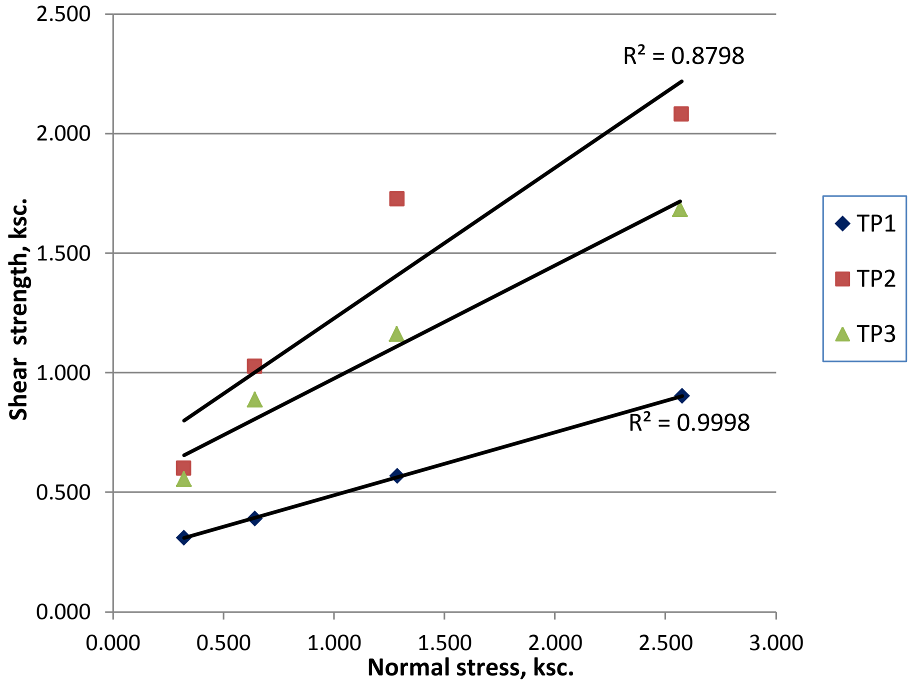
Test pit	WC %	Depth(m)	Degree of weathering	c (ksc.)	ϕ (Degree)
TP 9	11.97	3.0	Grade V	0.617	22.29

Appendix table B2 : Test results Multi-stage direct shear of Undisturbed sample at various degree of saturation or KU-MDS test (consolidation drained test)

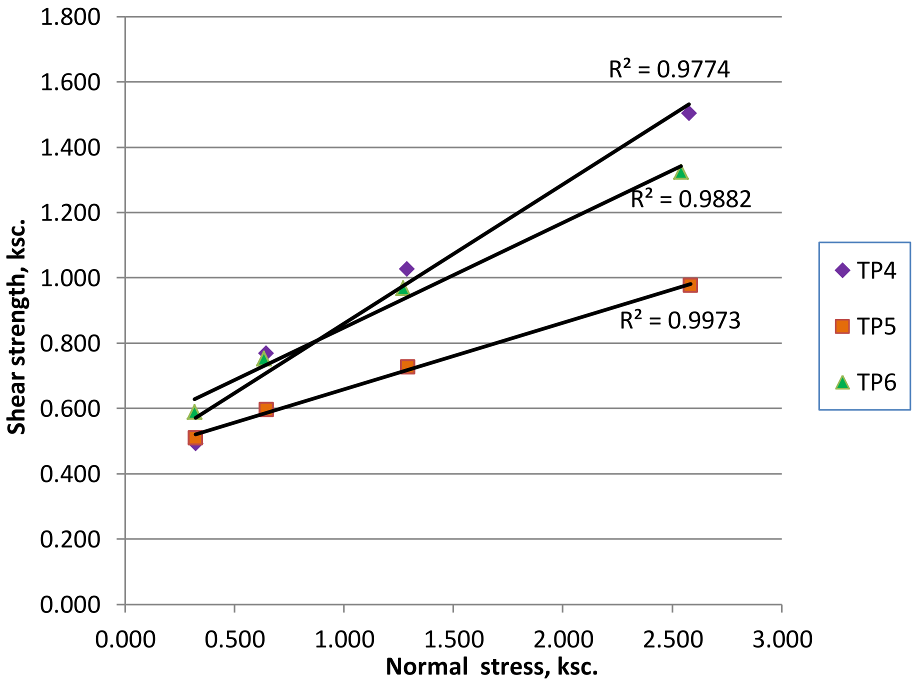
Degree of saturation, %	40 - 60		60 - 75		75 - 90		90 - 100	
	c'	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'
TP 1	0.31	20.47	0.28	12.33	0.22	14.77	0.19	21.58
TP 4	1.19	20.84	0.45	11.51	0.43	23.08	0.09	35.78
TP 8	0.40	21.9	0.71	15.17	0.32	13.08	0.14	19.79

Appendix table B2(continue) : Test results Multi-stage direct shear of Undisturbed sample at various degree of saturation or KU-MDS test (consolidation undrained test)

Degree of saturation, %	30 - 50		60 - 75		75 - 90		90 - 100	
	c	ϕ	c	ϕ	c	ϕ	c	ϕ
TP 9	0.61	22.28	0.53	29.8	0.43	19.45	0.18	25.34

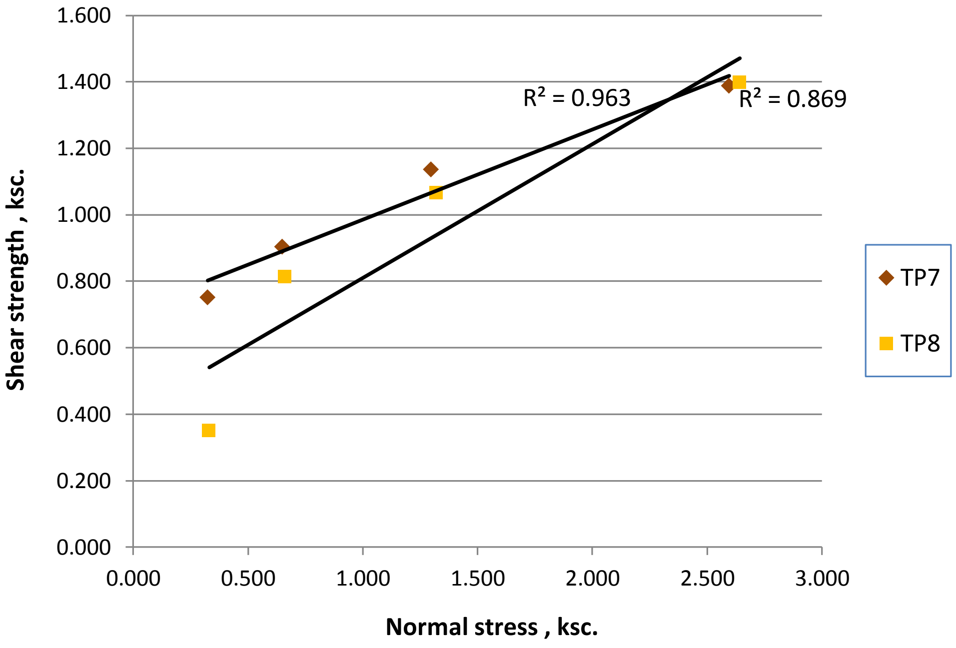


Test Pit TP 1, TP 2 and TP 3 (CD test)

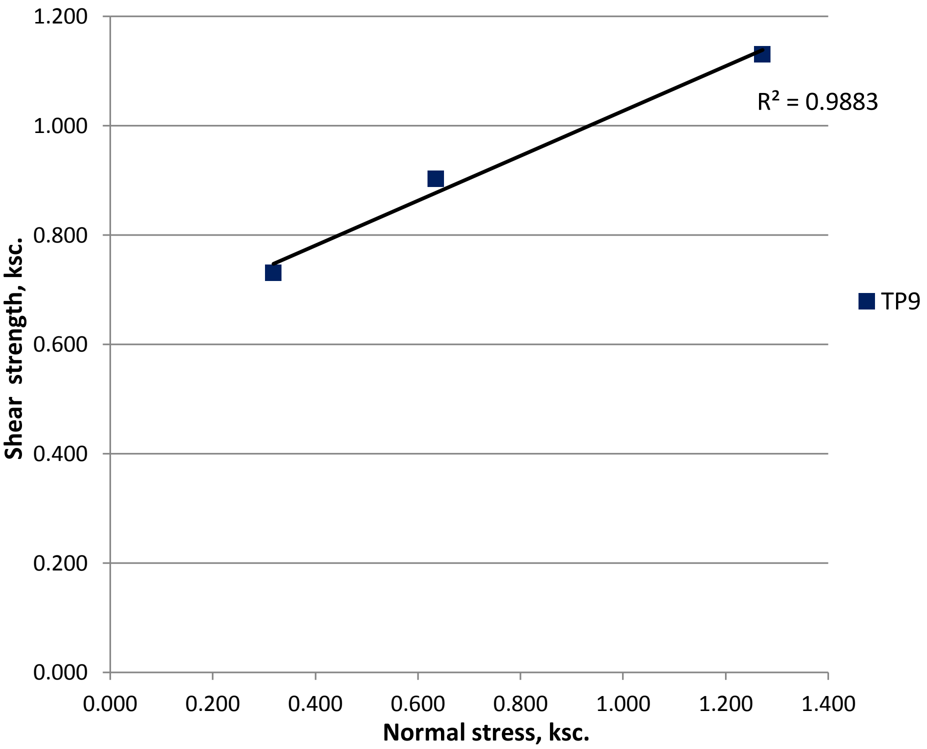


Test Pit TP 4, TP 5 and TP 6 (CD test)

Appendix figure B1 Testing results Direct shear (Multi-stage type)



Test Pit TP 7, TP 8 (CD test)



Test Pit TP 9 (CU test)

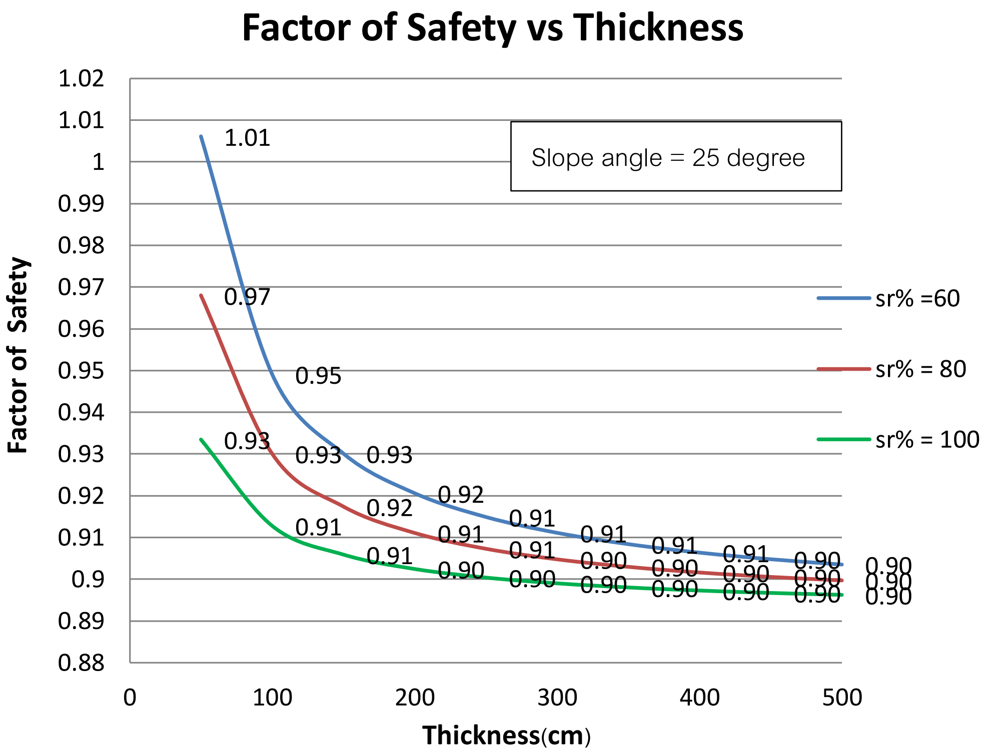
Appendix figure B1 (continue) Testing results Direct shear (Multi-stage type)

APPENDIX C

Results the Factor of Safety Analysis

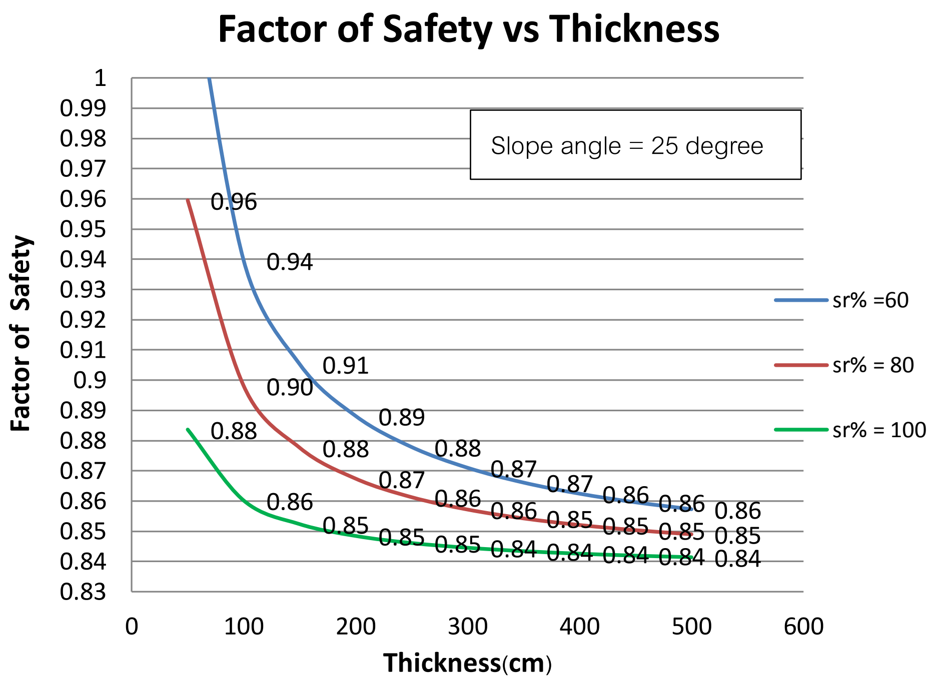
Results the Factor of safety analysis

From results the factor of safety analysis in each various Test Pit. The factor of safety value varies inversely with saturation value by which the range of critical saturation at approximate 90 % , 80 % and 70 . In addition to, factor of safety have value decreases according to the level of degree of saturation increases when soil slope have value slope angle is greater than or equal 25 degree, respectively. Summary testing result as shown in Appendix figure C1 (Factor of Safety and thickness), Appendix figure C2 (Factor of Safety and Slope degree)

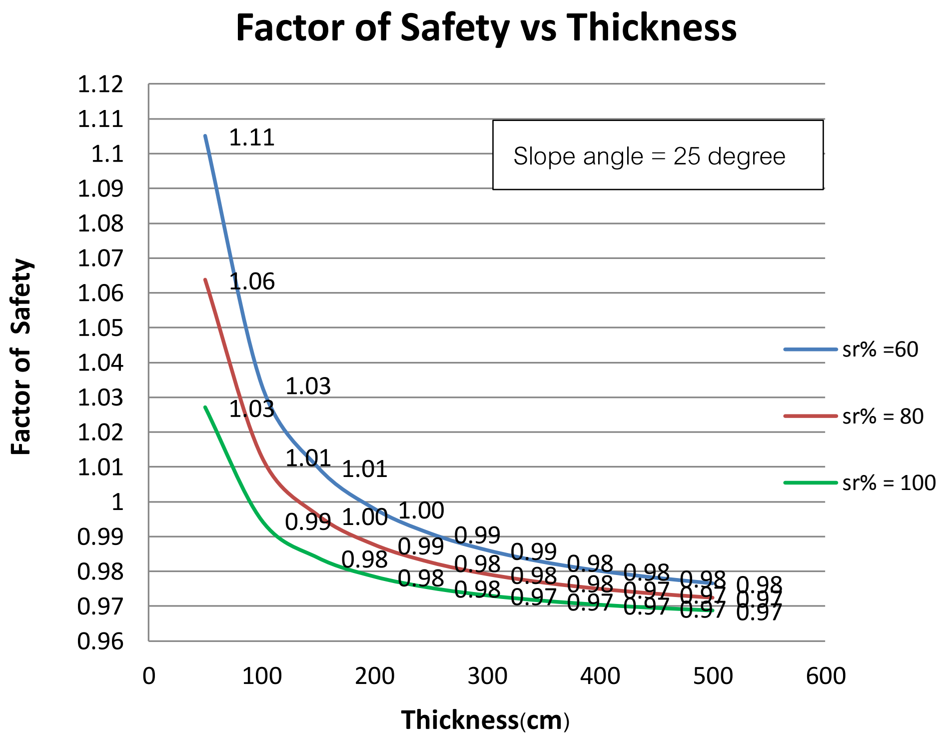


Test Pit TP 1 (CD test)

Appendix figure C1 Results the factor of safety with thickness of soil layer

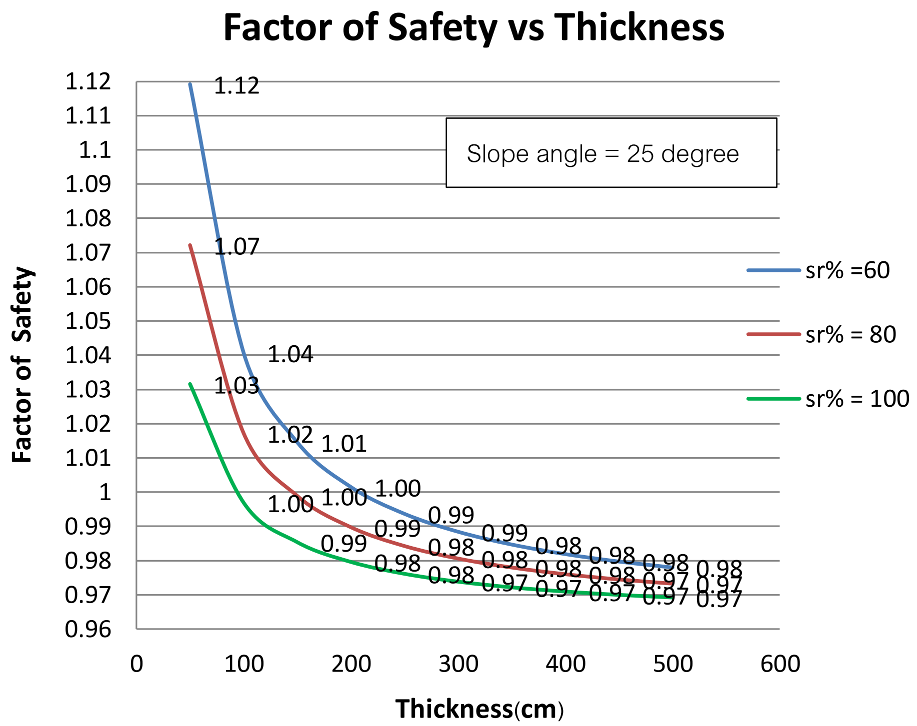


Test Pit TP 4 (CD test)



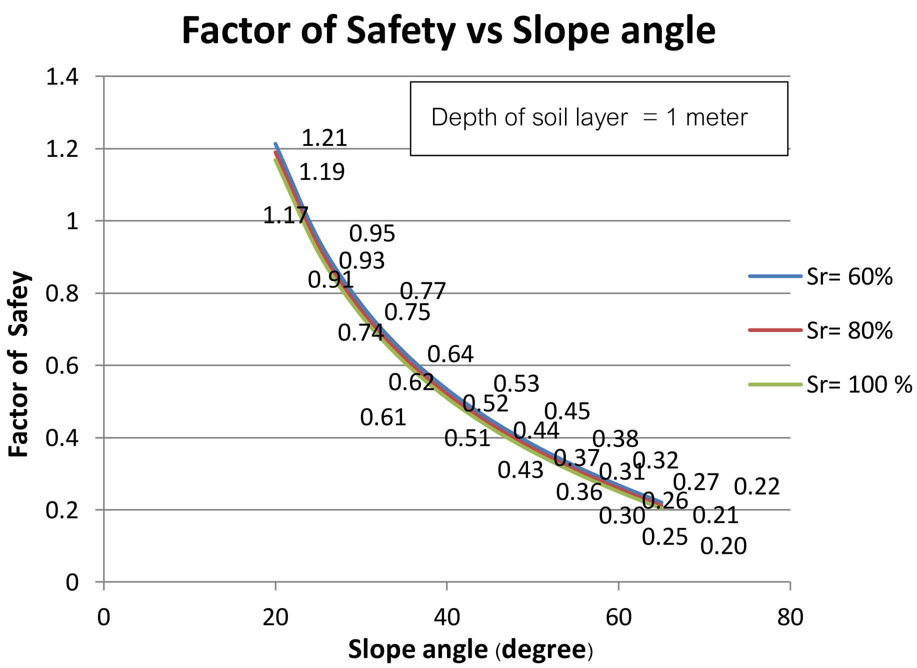
Test Pit TP 8 (CD test)

Appendix figure C1(continue) Results the factor of safety with thickness of soil layer



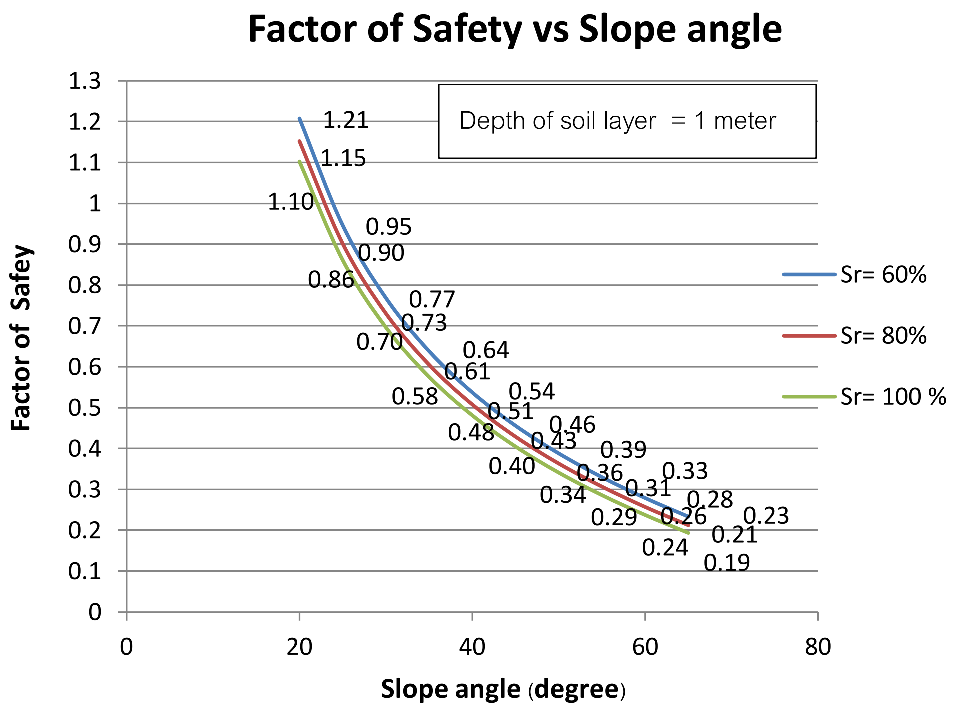
Test Pit TP 9 (CU test)

Appendix figure C1(continue) Results the factor of safety with thickness of soil layer

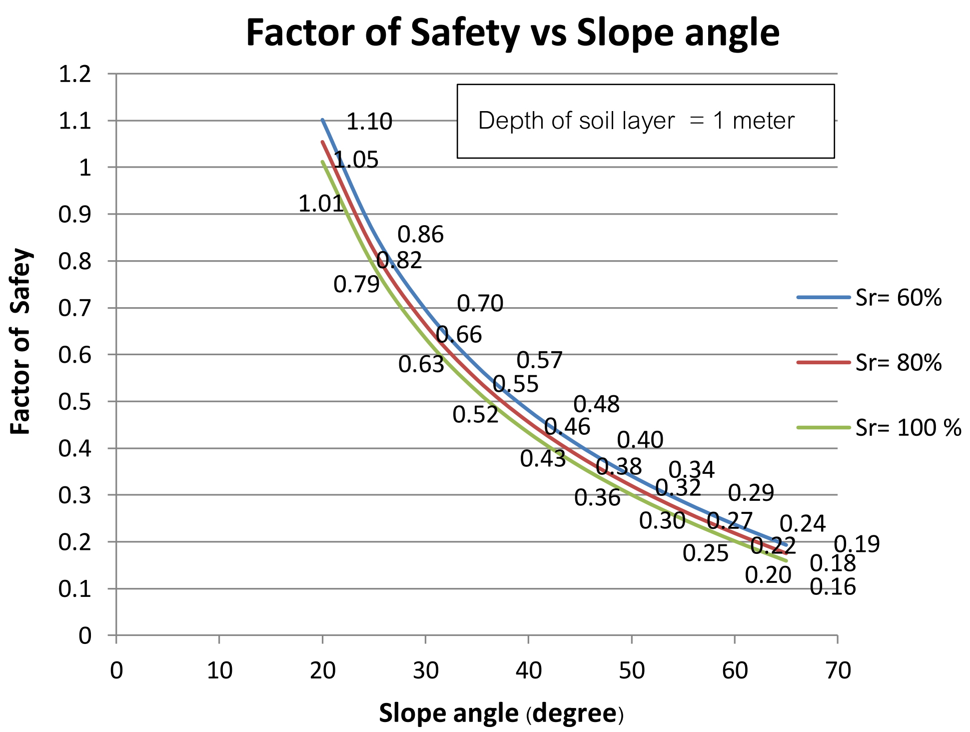


Test Pit TP 1 (CD test)

Appendix figure C2 Results the factor of safety with slope angle

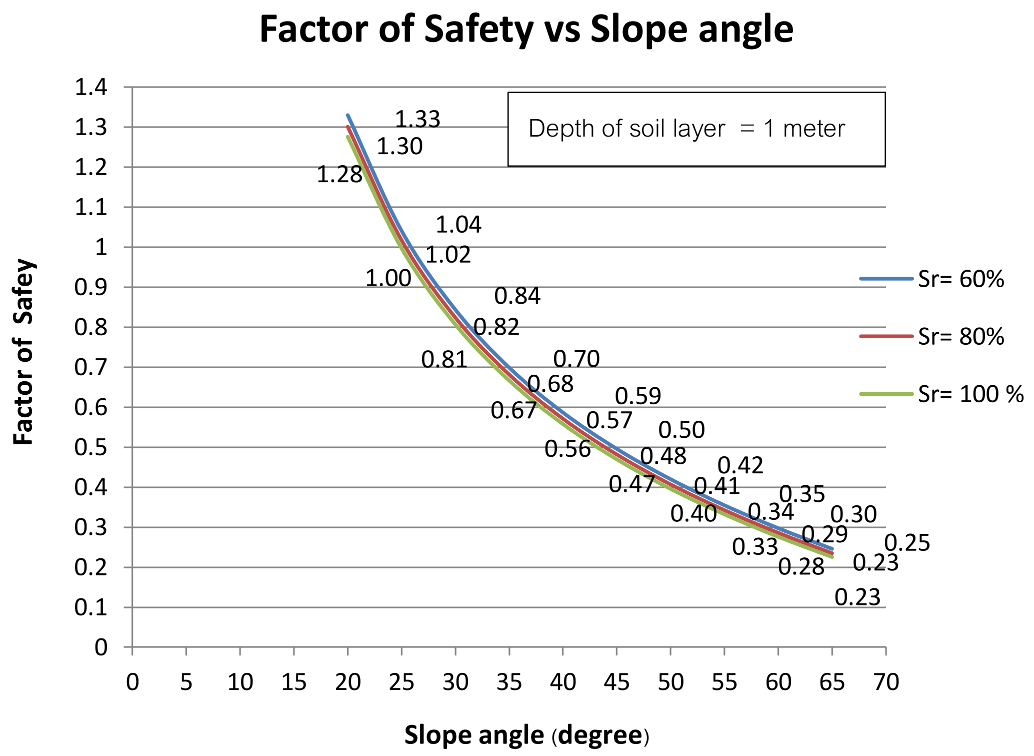


Test Pit TP 4 (CD test)



Test Pit TP 8 (CD test)

Appendix figure C2(continue) Results the factor of safety with slope angle



Test Pit TP 9 (CU test)

<u>Appendix figure C2(continue)</u> Results the factor of safety with slope angle

Biography

Mr.Thanakrit Thongkhao was born on October 23, 1982 in Nakhon Srithammarat Province, southern Thailand. He graduated high school level from Triam Udom Suksa School of the south, Nakhon Srithammarat Province, in 2001. In 2005, he received a B.Sc. degree in Soil Science from the Department of Earth Science, Faculty of Natural Resources, Prince of Songkhla University, Thailand. After his graduation, he started his work in 2005 as a sale and representative at Thai seed and agriculture company limited (Thailand). In 2009, he enrolled as a graduate student for M.Sc. program of Earth Science Programme from the Department of Geology, Faculty of Science, at Graduate School, Chulalongkorn University, Bangkok, Thailand.